Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO

CANMET

ON

REVIEW AND EVALUATION OF MAIN ROADWAY INTERSECTION TECHNIQUES FOR USE IN THE SYDNEY COALFIELD DSS.

CONTRACT SERIAL NUMBER 79017-01-SQ
CONTRACTOR: GOLDER ASSOCIATES
CONTRACT REQUISITION NUMBER 23440-7-9019

Distribution:

15 copies - CANMET
(Cape Breton Research Laboratory)
Sydney, Nova Scotia

4 copies - Golder Associates
Mississauga, Ontario

211 pp

MICROMEDIA

June 1988

871-1369

ABSTRACT

A review of the support design and practice for main roadway intersections in the United Kingdom, the United States, Canada, South Africa and Australia has been undertaken. The tradition of using steel supports still dominates coal mining in the United Kingdom, Europe and Canada. The roof support system in the United States is exclusively rock bolting. In South Africa and Australia, the current trend is to use rock bolts in the place of passive supports such as timber posts and steel arches. Steel sets are still used for poor ground conditions.

For most countries, the geotechnical design of either active or passive support systems for roadway intersections is still very much based on empirical methods and precedent experience/practice. The United States is the only country which attempts to conduct analyses, rock mass characterization and field observations to derive support (rock bolting) guidelines for roadway and roadway intersections.

The potential failure zone at the roof of a typical roadway intersection in the Lingan Mine has been analysed by back analysis from the estimated deflections of collar beams and roof beams, by stress analysis of the openings pertaining to the intersection geometry and by empirical design method. All the methods which take into account the rock mass characteristics give fairly consistent results. In comparison, the CBDC empirical design method tends to be conservative for the average condition at the Lingan Mine.

The average roof condition at Lingan Mine is such that roof bolting is a viable alternative support or a supplement to steel sections for the main roadway intersections. In

addition the increasing tendency to use heavier support in the operating mines in the Sydney Coalfield in response to the observed performance of the support should be re-examined by considering: the need to justify the support used; other factors which might have influenced the support performance but may not be directly related to rock loading; and the use of an alternative support system.

Recognizing the geological environment and the operating conditions in the Sydney Coalfield, a phased approach would be most appropriate for enhancing the current support design methodology and practices.

Phase 1 - characterization of selected existing intersections
Phase 2 - performance monitoring of a new intersection supported by current support systems
Phase 3 - implementation and monitoring of an experimental program to demonstrate the performance of viable support alternatives
Phase 4 - implementation and monitoring of a full scale intersection using the enhanced support system derived from phases 2 and 3

RESUME

La révision du design du soutènement et de son utilisation aux intersections des galeries principales des mines de charbon situées en Angleterre, aux Etats Unis, au Canada, en Afrique du Sud ainsi qu'en Australie a été entreprise lors de ce projet. L'utilisation des structures d'acier comme soutènement reste encore la plus populaire en Angleterre, en Europe ainsi qu'au Canada. Le soutènement utilisé au toit des galeries aux Etats-Unis est exclusivement constitué de boulon d'ancrage.

En Afrique du Sud et en Australie, la tendance actuelle est d'installer des boulons au lieu d'utiliser du soutènement passif comme des arches et des poutres de bois. Les arches d'acier sont cependant encore utilisés aux endroits où les conditions de terrain sont mauvaises.

Dans la plupart des pays, le design géotechnique du soutènement actif ou passif installé aux intersections des galeries principales est encore basé le plus souvent, sur des méthodes empiriques et l'expérience acquise précédemment. Les Etats-Unis est le seul pays qui essaie de combiner, la caractérisation du massif rocheux et les observations sur le terrain pour déterminer un guide du boulonnage s'appliquant aux galeries principales et aux intersections.

La zone de fracture potentielle, au toit d'une intersection typique de galeries principales à Lingan Mine a été analysée de la façon suivante: rétro-analyse de la flèche des différentes poutres de soutènement utilisées aux intersections, analyses des contraintes du massif rocheux occasionnées par la géométrie particulière des intersections ainsi qu'une analyse du design empirique du soutènement actuel. Toutes les analyses effectuées en

tenant compte des caractéristiques du massif rocheux ont donné des résultats, consistents. Cependant, la méthode de design empirique utilisée par CBDC semble être plutôt conservatrise pour les conditions moyennes du massif rocheux rencontré à Lingan Mine.

Les conditions moyennes du toit des galeries à Lingan Mine sont telles, que le boulonnage s'avère être une alternative possible ou supplémentaire aux arches d'acier utilisés à l'intersection des galeries principales. De plus, la tendance d'augmenter la grosseur des poutres utilisées comme support dans les nouvelles opérations dans les houillères de Sydney afin d'améliorer la performance du soutènement utilisé devrait être reconsidérée en tenant compte des points soivants; justification de l'emploi du soutènement actuel, analyse des autres facteurs qui peuvent influencer la performance du soutènement sans être pour autant être relié au mécanisme de chargement.

Afin de caractériser l'environnement géologique et les conditions d'opérations des houillères de la région de Sydney, une approche par étape subséquente semble être appropriée afin d'améliorer la méthodologie et la mise en pratique de la méthode de design du soutènement actuelle

Etape 1: Caractérisation de certaine intersection actuelle.

Etape 2: Mesure de la performance d'une nouvelle intersection supportée avec le soutènement actuel.

Etape 3: Implantation et évaluation d'un programme expérimentale démontrant la performance de d'autres mode de soutènement.

Etape 4: Implantation et évaluation d'une intersection supportée selon les modes de soutènement proposés à l'étape 2 et 3.

TABLE OF CONTENTS

	Page No.
ABSTRACT	i
1.0 INTRODUCTION	1
2.0 REVIEW OF ROADWAY INTERSECTION DESIGN	4
2.1 Design of Roadway Intersections in United Kingdom	4
2.2.1 Rock Conditions	4
2.1.2 Excavation Method	6
2.1.3 Support Type and Practice	7
2.1.4 Design Procedure	9
2.1.5 Design Implementation	11
2.2 Design of Roadway Intersections in the United States	12
2.2.1 Rock Conditions	13
2.2.2 Excavation Method	15
2.2.3 Support Type and Practice	16
2.2.4 Design Procedure	19
2.3 Design of Roadway Intersections in South Africa	30
2.3.1 Rock Conditions	30
2.3.2 Excavation Methods	32
2.3.3 Support Type and Practice	33
2.3.4 Design Procedure	36
2.3.5 Design Implementation	38
2.4 Design of Roadway Intersections in Australia	41
2.4.1 Rock Conditions	41
2.4.2 Excavation Methods	41
2.4.3 Support Type and Practice	42
2.4.4 Design Procedure	46
2.5 Summary	47
3.0 EXISTING CONDITIONS IN THE SYDNEY COALFIELD	50
3.1 Lingan Mine	50
3.1.1 Support for Main Roadways and Intersections	51
3.1.2 Support Performance	52
3.2 Prince Mine	55
3.2.1 Support for Main Roadways and Intersections	56
3.2.2 Support Performance	56
3.3 Phalen Mine	57
3.3.1 Support for Main Roadways and Intersections	57
3.3.2 Support Performance	58

TABLE OF CONTENTS (CONT'D)

	Page No.
3.4 Current Design Practice of Support System for Main Roadway Intersections in the Sydney Coalfield	59
4.0 REVIEW AND EVALUATION OF MAIN ROADWAY INTERSECTIONS FOR THE LINGAN MINE	61
4.1 Evaluation of the Support System for Roadway Intersections	63
4.2 Stability analysis of Roadway Intersections at Lingan Mine	67
4.2.1 Stress Analysis	68
4.2.1.1 Analysis of stresses and Deformation of Pillars at Intersections	68
4.2.1.2 Analysis of Potential Failure at the Roof of the Intersections	71
4.2.1.3 Discussion	76
5.0 APPLICATION OF ROADWAY INTERSECTION DESIGN TECHNIQUES TO THE SYDNEY COALFIELD	80
6.0 CONCLUSIONS	88
7.0 ACKNOWLEDGEMENTS	93
REFERENCES	94

LIST OF TABLES

Table 2.1 - An example of colliery support rules.

Table 2.2 - Properties of rock at a Illinois Basin mine (110 m)

Table 2.3 - Properties of rock at a Illinois Basin mine (293 m)

Table 2.4 - Summary of the review of roadway intersection design

Table 3.1 - Mechanical properties of rock in the Lingan, Prince and Phalen Mines

Table 3.2 - Stresses measured at different mines in the Sydney Coalfield

Table 3.3 - Properties of structural members used for intersection support for Lingan, Prince and Phalen mines

Table 4.1 - Estimated rock load on collar beam at a typical four-way intersection at the Lingan mine

Table 4.2 - Summary of deflections and maximum bending stresses in roof beams at a four-way intersection

Table 4.3 - Estimated rock load on collar beam at a typical three-way intersection at the Lingan mine

Table 4.4 - Summary of deflection and maximum bending stresses in roof beams at a three-way intersection

Table 4.5 - Summary of input parameters for NFOLD modelling

Table 4.6 - Summary of input parameters for boundary element modelling

Table 4.7 - Depth of potential overstressed zone at the roof of roadway intersection in the Lingan mine (Based on Elastic Boundary Element Analysis)

Table 4.8 - Mechanical properties used in finite difference modelling

Table 4.9 - Depth of potential overstressed zone at the roof of roadway intersection in the Lingan mine (Based on FLAC analyses)

Table 4.10 - Estimate of potential failure zone above main roadway intersections

APPENDIX B

Table B.1 - Formulae for maximum displacement and bending stress calculations

APPENDIX C

Table C.1 - Summary of the system of roadway support

Table C.2 - Summary of the boundary element analysis of a typical slope section

Table C-3 - Input parameters used in the evaluation of the elastic deflection and tensile stress at the centre of the arch

Table C-4 - Elastic deflections and tensile stresses calculated at the center of a typical arch at Lingan mine

LIST OF FIGURES

Figure 2.1 - Geology above the Deep Hard Seam British coal western area

Figure 2.2 - Fishplated junction arches

Figure 2.3 - Junction using fishplated supports

Figure 2.4 - Junction using flexible joints and yield boxes

Figure 2.5 - Typical four-way junction

Figure 2.6 - Junction support fabricated at the colliery

Figure 2.7 - Junction support fabricated at the colliery

Figure 2.8 - Example of trapezoidal support

Figure 2.9 - Example of types of strut

Figure 2.10 - Example of lagging

Figure 2.11 - Example of stilts

Figure 2.12 - Footpiece

Figure 2.13 - General columnar section of mine roof-rock (after Unal 1983)

Figure 2.14 - Active support - typical roof truss

Figure 2.15 - Rock bolting pattern (after Stahl 1962)

Figure 2.16 - Rock bolting support (after Gerdeen et al 1977)

Figure 2.17 - Loosened zone (after Peng et al 1978)

Figure 2.18 - Rock bolting pattern (suspension action) (Peng et al 1978)

Figure 2.19 - Rock bolting pattern suggested (reinforcement effect) (Peng et al 1978)

Figure 2.20 - Plot of the results obtained with photo-elasticity analysis (after Kaneshige et al 1964)

Figure 2.21 - Plot of the results from boundary-elements analysis (after Hocking et al 1976)

Figure 2.22 - Plot of vertical displacement (after Gercek 1982)

Figure 2.23 - Vertical stress distribution (after Gercek 1982)

Figure 2.24 - Area of special roof bolting pattern

Figure 2.25 - Roof bolting pattern guidelines (after Unal 1983)

Figure 2.26 - Suggested intersection support

Figure 2.27 - New truss bolting system

Figure 2.28 - Method of forming junction in South African bord and pillar mining

Figure 2.29 - Third generation support, Australia (after Hebblewhite 1986)

Figure 2.30 - Typical support pattern, third generation support

Figure 3.1 - General arrangement of three-way support Lingan mines

Figure 3.2 - General arrangement of four-way support Lingan mine

Figure 3.3 - General arrangement of four-way support Lingan mine

Figure 3.4 - Location map

Figure 3.5 - General arrangement of three-way support Prince mine

Figure 3.6 - General arrangement of four-way support Prince mine

Figure 3.7 - General arrangement of three-way support Phalen mine

Figure 3.8 - General arrangement of four-way support Phalen mine

Figure 3.9 - Plot of extensometer results (Phalen mine)

Figure 3.10 - Assumed configuration of the rock loads for the design of arches and intersection support - CBDC Design Practice

Figure 4.1 - Hypothetical Load - Deformation curves for the rock mass and the support

Figure 4.2 - Typical loading configurations for collar and roof beams

Figure 4.3 - Plot of the different bending stresses caused by equivalent uniform load for the roof and collar beams (four-way intersection)

Figure 4.4 - Plot of the different mid span deflections caused by equivalent uniform load for the roof and collar beam (four-way intersection)

Figure 4.5 - Plot of the different bending stresses caused by equivalent uniform load for the roof and collar beam (three-way intersection)

Figure 4.6 - Plot of the different mid span deflections caused by equivalent uniform load for the roof and collar beam (three-way intersection)

Figure 4.7 - Mining pattern - Lingan Mine typical three-way "T" intersection

Figure 4.8 - Mining pattern - Lingan Mine typical four-way intersection

Figure 4.9 - Convergence pattern - Lingan Mine typical three-way "T" intersection

Figure 4.10 - Convergence pattern - Lingan Mine typical four-way intersection

Figure 4.11 - Normal stress pattern - Lingan Mine typical three-way "T" intersection

Figure 4.12 - Normal stress pattern - Lingan Mine typical four-way intersection

Figure 4.13 - Finite difference grid showing typical geology of Lingan Mine

Figure 4.14 - Stress trajectories around an intersection

APPENDIX A

Figure A.1 - Lingan Mine site visit location plan

Figure A.2 - Typical stratigraphy of Lingan Mine and typical rock properties

APPENDIX C

Figure C.1 - Plot of theoretical elastic deformations and tensile stress of an arch uniformly loaded

LIST OF PLATES

Plate 1.A - Lingan mine
4-way intersection (point 1 fig. 3.4)
4 slope, 2 east bottom

Plate 1.B - Lingan mine
4-way intersection (point 1 fig. 3.4)
4 slope, 2 east bottom

Plate 2.A - Lingan mine
4-way intersection (point 2 fig. 3.4)
4 slope, 3 east bottom

Plate 2.B - Lingan mine
4-way intersection (point 2 fig. 3.4)
4 slope, 3 east bottom

Plate 3.A - Lingan mine
4-way intersection (point 3 fig. 3.4)
4 slope, 4 east top

Plate 3.B - Lingan mine
4-way intersection (point 3 fig. 3.4)
4 slope, 4 east top

Plate 4 - Lingan mine
4-way intersection (point 4 fig. 3.4)
2 slope, 5 west bottom

Plate 5.A - Lingan mine
4-way intersection (point 5 fig. 3.4)
3 slope, 10 east top

Plate 5.B - Lingan mine
4-way intersection (point 5 fig. 3.4)
3 slope, 10 east top

Plate 6.A - Lingan mine
3-way intersection (point 6 fig. 3.4)
2 slope, 9 east top

Plate 6.B - Lingan mine
3-way intersection (point 6 fig. 3.4)
2 slope, 9 east top

Plate 7 - Lingan mine
3-way intersection (point 7 fig. 3.4)
2 slope, 10 east top

Plate 8 - Lingan mine
3-way "Y" intersection (point 8 fig. 3.4)
4 slope, 6 east bottom

Plate 9 - Lingan mine
3 way "Y" intersection (point 9 fig. 3.4)
2 slope, 4 west bottom

LIST OF APPENDICES

A. REPORT ON UNDERGROUND VISIT TO LINGAN MINE

B. FORMULAE FOR MAXIMUM DISPLACEMENT AND BENDING STRESS CALCULATIONS

C. ANALYSIS OF ARCH STABILITY IN MAIN ROADWAYS AT LINGAN MINE

1. INTRODUCTION

Roadway support design in coal mines are generally based on empirical design procedures or precedent practices in the same mining environment. However, as underground coal mining involves progressively deeper depths and more complex mining layout, roadways and, in particular, roadway intersections should be designed with emphasis on rational design procedures and optimization.

Roadways and their intersections are often driven either in coal or in the adjacent strata which invariable comprise layers of sandstone, siltstone, shale, mudstone and seatearth.

The stability of underground excavations is governed by the geometry of the excavation, the magnitudes and orientations of the in situ stresses, the mechanical properties of the rock and the geological structures. The formation of the roadway will cause a redistribution of the initial stresses in the rock. The state of stress acting on the rock elements surrounding the roadway will vary according to their locations with respect to the excavation boundaries. Close to the excavation boundary, the rock would essentially be subjected to a biaxial stress system. Further away from the excavation boundary, confinement from adjacent rock mass would give rise to a triaxial stress state.

The stability of the roadway would depend on whether the rock can sustain the new stress state. If the rock mass strength (generally lower than the strength based on intact laboratory specimens) is not sufficient to withstand the stresses, rock failure would be initiated. If rock support is not provided or delayed, progressive failure would take

place. This would be manifested in the form of rock fracturing and would subsequently lead to rockfalls. In coal measures strata, the bedded nature of the deposit is also conducive to bedding plane separation and buckling of the roof strata. In outburst-prone strata (such as that seen at No. 26 Colliery Sydney Coalfield), a combination of gas effects and rock overstress (plus the dynamic stresses due to blasting) could lead to instantaneous rock and gas outbursts.

The support used for main roadways in the Sydney coalfield comprises mainly steel arches. At roadway intersections, assemblies of heavy I beams are generally used. These passive support systems however, would allowed progressive fracturing of the rock to develop at the roof until sufficient reactive pressure is derived from the support to provide the necessary confinement to stop further rock fracturing/loosening. Field observations have indicated that some structural (beam) members have shown excessive deflections and that substantial rock loosening was clearly evident at the roof.

While steel arch supports are traditionally used in the Sydney coalfield as well as in the coal mines in the United Kingdom and Europe, rock support in the U.S. Coal mines is almost exclusively rock bolting. The current trend in the U.K. coal mines, however, is to examine the use of rock bolt as an alternative support system.

Similarly, the Cape Breton Development Corporation is also interested in considering the rock bolting concept as an alternative support for main roadways and roadway intersections. In connection with this, Golder Associates was awarded a contract (DSS Contract No. 23440-7-9019/01-SQ) under the auspices of the Cape Breton Coal Research Laboratory of CANMET to review and evaluate roadway intersection support techniques for use in the Sydney coalfield.

The study included:

- A review of the roadway intersection techniques used in coal mines in different countries.
- A review and evaluation of the existing designs and rock conditions for roadway intersections in the Sydney Coalfield, with particular reference to the Lingan Mine.
- Consideration for applying alternative intersection design methods to the Sydney coalfield.

The following sections present the results of the review and evaluation, and the conclusions reached.

2. REVIEW OF ROADWAY INTERSECTION DESIGN

Support for development roadways and their intersections is probably one of the most important parts of the day to day colliery activities. Depending on the mining methods used and the rock conditions encountered, different support systems are used for roof control in coal mines in different countries.

A review of the roadway intersection design procedure and practice adopted by different countries including the United Kingdom, the United States, South Africa and Australia is presented in this section of the report. The support practice in the Sydney Coalfield is discussed separately in Section 3.

2.1 Design of Roadway Intersections in United Kingdom

It has proved very difficult to obtain any publications referring to the design of intersection support used throughout British Coal. In order to illustrate the general methodology which has evolved in connection with intersection support, some examples from the Western Area were used for this study.

2.1.1 Rock Conditions

The geology of the coal mining areas varies significantly throughout UK. The areas of greatest production exist between North Yorkshire in the north and the SE Nottingham and Leicestershire areas to the south where working seams are generally in excess of 300 m deep and can be up to 1000 m deep in places. Most mining occurs in multi-seam areas, with the current mining generally not the first extraction. Consequently working faces are often associated with some degree of previous extraction.

In general, the geology of the immediate seam consists of mudstones, siltstones, or fine grained sandstones. The degree of lamination can vary from seam to seam, or within the same seam and is often related to the nature of the roof geology. The roof of most seams is characterized by siltstones with minor mudstones and fine grained sandstones. Because of the autochthonous (not transported) nature of the coal in the UK, many seams are underlain by a weak often expansive mudstone which is called a seatearth. This material is highly sensitive to stress and often to water.

In many seams, the thickness of the coal is less than the height needed to allow passage of men and materials. Consequently some form of roof dinting or roof brushing is practised. In such situations, the face geology of the roadway will consist of the coal seam from the floor level but will change to include part of the roof sediments. The fact that the roof strata may be cut or blasted will have implications on the stability of the openings, especially in the upper ribsides.

At some distance above the seam, the coal measures in UK are often overlain by water-bearing strata, either of the Permian sequences, or of the Magnesian Limestone. There are statutory limits specifying the minimum thickness of non-permeable strata which must be present between the worked seam and the base of the aquifer.

The limited data available in Britain regarding in-situ stress indicates that the vertical stress is a principal stress and is related to the depth in the usual form

$$\sigma v = \gamma \times D$$

where σv is the vertical stress in MPa and D is the depth of the seam in metre and γ is the density in Mg/m^3.

In the past two or three years, the attention of many workers has concentrated upon the general virgin stress prevalent in the mining areas. There is a growing belief in the UK that the active mining areas are characterized by a horizontal stress in excess of the vertical stress. In addition, the horizontal stress is believed by some to be anisotropic in nature, and Altounyan and Bigby (1988) have reported that the higher horizontal stress component is orientated NW-SE, and can be as much as twice the lower horizontal stress. This directional trend can be identified underground by studying the development of shear failure in the roof of suitably oriented roofways.

For many years, it has been an accepted tenet that in some areas there is a directional preference to underground stability. This may be the result of cleat in the coal or jointing in the roof rocks but may also be influenced by the degree of any anisotropy in the horizontal stress regime.

Figure 2.1 shows the geology of the area of the Deep Hard seam which occurs at a depth of about 425 m in the Western Area and has a thickness of about 1.7 m. The immediate roof of the seam consists of mudstone, but the succession above the seam for at lest 100 m consists of alternating coal, mudstones, siltstones and some sandstones, and is typical of the Upper Coal Measures in general.

2.1.2 Excavation Method

Most roadways are excavated by roadheaders of some description. The use of a road header often permits a certain amount of roof profiling to be carried out in order to relate the roadway profile to the steelwork likely to be installed, but it is not the case that the face profile is deliberately shaped to match the proposed supports. It is

likely that a certain amount of over-mining takes place especially to facilitate the installation of the steelwork.

2.1.3 Support Type and Practice

Figures 2.2 to 2.5 illustrate the types of passive supports which are supplied by manufacturers to comply with the needs of local collieries. Where flat topped roadways are cut, then the colliery will make up its own sets. Arched supports are generally purchased from specified approved suppliers, and the colliery is obliged to cut a roadway profile to suit the size of arch which is available. Junction arches are available ranging in width from 5.22 m to 12.69 m and in heights of 3.86 m to 4.89 m, where the sizes quoted apply to the inside clearances of the supports.

Flat topped development is usually supported by steel sets fabricated in the colliery. Standarized sections of RSJ are usually used, with individual beams and stanchions formed by welding together smaller sized RSJ sections. Methods of joining the beams to the legs of stanchions vary from place to place. In general, the method of dealing with wider intersections using locally fabricated sets is to produce heavier members by welding together lighter RSJ sections (see Figures 2.6 and 2.7).

A third general type of passive support in use for flat-topped roadways is the trapezoidal square work illustrated in Figure 2.8. This support is a manufactured system supplied at a number of widths and heights, and sometimes used for intersection support.

To all of the systems of passive support, a number of accessories are commonly used, or required by the support rules.

i) Struts (see Figure 2.9)
These are used to give lateral stability to the supports by linking each set to those adjacent to it. The strut length is also used to control the spacing between individual sets, as this spacing is often decreased to produce a heavier support system.

ii) Lagging (see Figure 2.10)
Lagging is fixed to the supports to prevent falls of ground between the arches. The materials used include wooden cover boards, corrugated steel sheets, wire mesh panels, and a variety of split or solid timber poles.

iii) Packing
Packing usually consists of timber which is used to fill voids between the supports and the final rock profile. In reality, packing is usually only utilized in areas of local instability where cavities have formed. It is unlikely that timber will be used to pack voids produced by over-mining of the roadway. The use of lagging, which often precludes inspection behind the supports, tends to lead to minimal standards of packing.

iv) Stilts (See Figure 2.11)
Stilts are used on the feet of supports in areas where it is believed that sets will be punched into the soft floor. Stilts are short lengths of steel or timber attached to the legs of the support by clamps which allow the yielding of the set either by sliding of the clamps or by crushing of the stilt.

v) Footpieces (See Figure 2.12)
Footpieces comprise large timber slabs which are used in an attempt to prevent penetration of the floor by the arches.

Active supports in the form of grouted rock bolts are slowly gaining acceptance within the UK coal industry. In the few instances where roof bolting has been used, the installation generally consist of resin grout capsules into which 20 mm to 25 mm diameter bolts, made from threaded reinforcement bars, are spun. Most of the installations are of an experimental nature and little information is yet available regarding bolting performance, especially at intersections. Although the use of roof bolts is slowly being accepted by the industry, at present their use can only be described as experimental or in response to very adverse conditions or on the face salvage line.

The timing of the installation of supports is governed by legislation, though much of the legislation specifies the support spacing for longwall faces and for the advanced gate roads. Local support rules govern the installation of girder or arch support at intersections, an example of which is presented as Table 2.1. In general, the regulations indicate that the last row of support should not be more than about 1.5 m from the face. This will, however, depend upon the support spacing being used.

2.1.4 Design Procedure

Discussions have been held with a number of British Coal engineers, in an attempt to ascertain the methods used to design supports. Without exception, British Coal engineers indicated that the design of passive supports, especially at intersections, is empirical and based upon precedence.

The actual layout of the supports to be installed is generally done by the colliery surveyor, who assesses the requirement based upon the success of installations in the vicinity. Where most difficult conditions are suspected or encountered in a given area, the steel sets are strengthened or the spacing decreased to increase the support density.

In the early 1950s the load carrying capacity of steel sets was evaluated from simple beam theory, apparently with the loading condition of a distributed load. Unfortunately, it has not been possible to ascertain whether or not any of the results were published.

No other work is known within British Coal, and no codes of practice appear to exist which govern support of intersections. Each colliery or Area appears to use its own empirically derived designs and standards depending upon the seam and the localized conditions. It has been reported verbally that the cost of supporting an intersection in British Coal is of the order of £30,000 ($72,000).

The design of roof bolting patterns is currently receiving a great deal of alteration in UK at present. In a paper reviewing the approach of British Coal to the problem, Morris and Newson (1988) describe some empirical and analytical approaches currently being used. Most of the methods cited were either simple evaluations based on the formation of a zone of compression around the bolts (eg. cited in Morris and Newson) or empirical methods such as the French Cerchar computer data base also discussed in Morris and Newson. Morris and Newson outlined the philosophy of understanding the geomechanical conditions and employing suitable instrumentation methods when attempting to design the optimum roof bolting support system. Whittaker and Scoble (1980) described a similar approach to roof bolt design albeit employed in a UK ironstone mine.

Nevertheless, no specific approach has yet been recorded indicating the design of bolted support for intersections in British coal mines.

2.1.5 Design Implementation

The sequence of installation of the individual components of steel sets is logical, and the local support rules (Table 2.1) usually specify the forms of temporary support required and the methods of erecting each set.

Similarly, the distance from the face to the first row of supports is governed by both legislation and the local support rules.

One of the principal limitations of using a passive support system of the type typically used by British Coal, is the tendency to overmine the excavation in order to facilitate easy installation of the supports. Once the lagging is installed it is very difficult to inspect the rock and to determine that there is adequate packing.

A further limitation to the method of design by precedence is that supports in general use tend to become heavier in response to problems which may be confined to local areas. Unless the variations in conditions are observed and understood the design process becomes a progressive intensification of support.

2.2 Design of Roadway Intersections in the United States

The United States in one of the largest producers and consumers of coal in the world. Over 90% of the underground coal production is by the room and pillar method. The use of longwall mining is rather limited. The average depths of coal mining is between 90 and 500 m below ground surface. In room and pillar mining, there are numerous entries and crosscuts which form intersections. Two kinds of intersections are present in the mines: (a) the T or Y intersections, composed of two or three roadways respectively and (b) the cross or X intersections formed by four roadways.

Statistically, 31 percent of the total number of fatalities occurred at intersections in the United States (Stahl, 1962) between 1956 and 1961 and between 1977 and 1988 (Peng. 1980). More recently, Unrug et al (1984) reported that between January 1980 and May 1982 about 29% of the total number of accidents occurred at intersections. These statistics highlight the importance of strata control and ground support at intersections and has indicated a demand for conducting research studies to improve the roof stability.

A lot of factors would influence the roof stability of an intersection. Since the 1950's, the United States Bureau of Mines (USBM) and other research institutions have been conducting research projects relating to the stability of intersections in coal mines and the methods of support. The work included empirical evaluations and numerical modelling of the stress distribution at the intersections (Gercek 1982) and a series of instrumentation at selected intersections in several coal mines to measure rock loads and changes in in situ stresses due to the room and pillar mining activities. (Haramy and Hanna 1985, 1988)

The use of Geomechanics classifications was also proposed by Unal (1983), for the design of rock bolting patterns to support the entries and roadway intersections.

The following sections present a review of the rock condition, the support design and the related research studies for the roadway intersections.

2.2.1 Rock Conditions

The United States has a large reserve of coal deposits distributed throughout the country. However, these deposits have a lot of common features in their structures and formation histories. In general, the roof of a typical U.S. coal mine can be characterised by a series of horizontal bedding planes, joints and other planes of weakness. The "immediate roof" overlying the workable coal seam generally consists of a sequence of thinly laminated beds. The coal measures strata generally comprise shale, siltstone, sandstone and coal, graded horizontally as well as vertically.

In some mines a shale or soft shale, sometimes called drawslate, occurs immediately above the coal and will fall when the coal is removed. Usually, the strata immediately above the roofs tend to be more thinly bedded than the overlying sandstone or limestone of the main roof. Sometimes the first bed over the coal seam is a relatively weak claystone or mudstone. The floor strata generally consist of sandstone, shale, mudstone, claystone or occasionally limestone.

Figure 2.13 shows a generalized stratigraphic section of the mine rock-roof in western Pennsylvania.

Tables 2.2 and 2.3 present some typical properties of the strata in two mines located in the Illinois basin. The

first mine is situated under shallow overburden (110 m) whereas the second is located under medium overburden (293 m).

Over the years, the rock types of coal mine roof have been investigated and categorized (Unal 1983). The rock characteristics associated with the roof quality are presented below:

Roof Quality	Rock Type
very poor roof	- slump deposit - channel scours - fire clay - kettlebottoms
poor to fair roof	- crossbedded and pebbly sandstone - sandstone with shale streaks and interbedded shale and sandstone - crystallized sandstone and conglomerate
good roof	- massive grey sandstone - massive sandy shale

Furthermore, it was observed that the presence of special geological conditions such as bedding planes, slickensides, rider seam, channels (paleochannels and scours, similar to those observed in the Sydney coalfield), pinchouts, clay veins and joints play a very important role in the roof stability.

On the other hand, one of the major parameters affecting the roof behaviour are the in situ field stresses. The

virgin stresses in a rock mass before any excavation is made and their relation to the boundary stresses resulting from excavation of the openings play an important role in roof stability.

In an effort to obtain a better understanding of roof fall occurrences, rock strength and ground stress were determined in the roof of several mines across Canada and the U.S. (Unrug 1984). Two aspects were highlighted by the measurements:

- Horizontal stresses exceed vertical stresses
- Maximum principal compression stresses are oriented preferentially in the E-W to NE-SW directions

However, factors such as geometry of excavations, method of excavation, environment conditions would also have an important effect on the roof stability.

A more detailed discussion on these and other factors will be presented in a later section of the report.

2.2.2 Excavation Method

Essentially, there are three mining methods used in underground coal mining in the United States, namely

(1)	Room and pillar mining	91%
	a) conventional (drill and blast)	11%
	b) continuous (Machine cutting)	80%
(2)	Longwall mining	8%
(3)	Shortwall mining	1%

For mine entries, continuous miners are frequently used as highlighted in the following table (Unal 1983).

Mine	Location	Type of Roadway	Mining Method	Seam Depth (m)	Seam Thickness (m)
A	Pennsylvania	mine entry	continuous/ longwall	90	1.2-1.5
B*	Pennsylvania	mine access	continuous	90-180	1.1-1.5
C	Pennsylvania	mine access	continuous	90-180	1.1-1.5
D	Pennsylvania	mine entry	continuous/ longwall	180	1.2-1.4
E	Pennsylvania	mine entry	continuous/ longwall	150	1.1-1.3
F	Pennsylvania	mine entry	continuous longwall	150	1.1-1.7
G	West Virginia	mine entry	continuous/ longwall	80-100	1.5-1.7

*tunnel boring machine used

Since the 1940's and 1950's significant efforts were made to build American-made continuous machines with a great advance rate.

Three types of machine were developed during the last twenty years, tunnel boring machine (TBM) boom-type roadheader and continuous miner. The utilization of these machines, influences directly the roof stability and permit the roof support system to be installed close to the excavation face.

2.2.3 Support Type and Practice

The roof support system for roof and pillar coal entries in the United States mainly comprises rock bolts. According to the statistics reported by Unal (1983), over 130 million bolts are used annually in the United States.

The supports used consisted mainly of different types of rock bolts (including conventional mechanically anchored bolts, fully resin grouted bolts and a combination of these) used in combination with cross beams, wooden posts, cribs, cross bars and steel beams. Roof truss assemblies comprising inclined rock bolts and tie rods have also been used at intersections (see Figure 2.14).

In the earlier years of coal mining in the United States, the roof support system were often selected on the basis of trial-and-error method, and previous experience of the mining engineers. Resin grouted rock bolts were often used and the length of bolts were chosen based on experience.

For example, the following patterns were used in two different mines but the additional support was approximately the same.

1st pattern	- rib bolts	- vertical 9 ft. long (expansion shell)
	- centre bolts	- vertical 5 ft. long (expansion shell)
	- 7 ft. or longer (resin)	
2nd pattern	- 3 ft. to 8 ft. bolts with expansion shell alone or with a combination of expansion shell and resin bolts	
Additional Support	- Truss bolts - Wooden post - Steel - Cribs - Wire Mesh	

No general agreement of the bolting pattern seems to be evident across the coal mines but the bolting practice in

the U.S. coal mines is generally in accordance with the Code of Federal Regulations, Title 30 presented below (imperial units are used in the Regulations). Sometimes the openings are lined with concrete and shortcrete and supplemented with steel sets but usually this was reserved for long-term openings such as main haulageway or shaft stations.

Code of Federal Regulations, Title 30

The following code of mining regulations was utilized in the assessment of design guidelines for support selection of coal-mine roofs.

"Subpart C - Roof Support

75.200-7 Criteria--Full roof bolting plan.

A full roof bolting plan is one in which roof bolts constitute the sole means of roof support at a face as part of the normal mining cycle.

(a) Roof bolt assemblies should meet the following specifications:

(1) All components of the roof bolt assembly should comply with the American National Standards Institute, "Specifications for Roof Bolting Materials in Coal Mines".

(2) Roof bolts that provide support by creating a beam of laminated strata should be of a length that assures adequate anchorage, but in no case should the length of the bolt be less than 30 inches.

(3) Roof bolts that provide support by suspending the immediate roof from a stronger overlying strata should be of a length that permits anchoring at least 12 inches in the stronger strata.

(b) Installation Practices:

(1) Torque ranges specified in the roof control plan should be capable of providing roof bolt loads to within plus or minus 1,000 pounds of 50 percent of either the yield point of the roof bolt being used or the anchorage capacity of the strata, whichever is less. In no case, however, should installed torques provide loads that exceed the yield point of the roof bolt being used or the anchorage capacity. Relationship for determining roof bolt load for torque applied are as follows:

(c) Roof bolting pattern:

(1) Roof bolt spacing either lengthwise or crosswise should not exceed 5 feet.

(2) Roof bolts should be installed as close as possible to, but not more than 5 feet from the rib before a sidecut is started.

(3) Roof bolts should be installed as close as possible to, but not more than 5 feet from, the face before starting conventional cutting or a continuous miner run.

(4) Openings should not exceed 20 feet in width where roof bolting is the sole means of roof support."

"75.200-9 Criteria--Combination roof-control plan

For a plan where both roof bolts and conventional supports are used for roof control at the face, the criteria for a Full Roof Bolting Plan and a Conventional Roof Control Plan should apply with the following modifications:

(1) Any place being driven over 20 feet in width should be supported by a Combination Roof Control Plan.

(2) The roadway should be limited to 16 feet in width on both the straight and the curves to within 10 feet of the uncut face.

(3) A row of posts should be set for each 5 feet of space between the roadway posts and the ribs.

(4) Openings should not exceed 30 feet in width."

2.2.4 Design Procedure

For many years, researchers have endeavoured to optimize the procedure for designing the support system for coal mine. Three approaches are currently used in the United States: empirical, observational and analytical. These approaches are also used in combination to arrive at design guidelines. In addition, the geological information and the mining regulations are also considered.

In the following sections, the theories which were used across the U.S.A. for the analysis of the intersection stability and design of the support system are reviewed and discussed.

Empirical and Semi-Empirical Approach

Stahl (1962) was the first to suggest some precautions regarding the support of junctions even before the 1969 Coal Mine Health and Safety Act was put in place for U.S. coal mines. Based on his review of roof falls, he proposed some rock bolting patterns and support systems for intersections as shown in Figure 2.15. These designs were based on the use of mechanical excavations. The proposed designs feature pattern bolting with longer bolts at the centre and edges as appropriate, inclined bolts at trimmed corners, posts at the entrance and cribs at the center of unused intersections, and occasional beams at the approach to an intersection.

One of the first approaches which was suggested by Obert et al (1960) to estimate the stress conditions of a laminated roof at intersections was to use the plate theory based on the mechanics of solids when the length-to-span ratio was less than 2. It offers a reasonable approach to evaluate the stress condition at the intersection when the immediate roof could be approximated as a plate deforming under some kind of distributed load and under its own weight.

Similar approach was also used by Wright (1973) who derived some multiplication factors that could be applied to the beam solution to give an approximation of stresses in the roof slabs spanning the intersections.

Gerdeen et al (1977) included intersections in their study involving design criteria for roof bolting plans using fully

resin-grouted non-tensioned bolts to reinforce bedded mine roof. The following suggestions were derived from their work.

- If vertical bolts can provide an adequate suspension of the roof from competent rock area for entry, the same approach with larger bolts can be used for intersections.

- If angled bolting over the ribs is an effective suspension for the entries, longer and larger bolts should be used at each corner of the intersection.

- Some posts or cribs could be used to complete the pattern.

- In the case of high horizontal stresses, a greater bolt density is needed over the intersection to prevent buckling of individual layers.

- If there is a predominant slip direction in the roof, a combination of planks and roof bolts adequately oriented with respect to the slips may be required to prevent roof falls (figure 2.16).

- When the horizontal stress is low in one direction, and high in the other, the predominant ground arch will form in the direction of low lateral stress. At an intersection, this arch disappears (Figure 2.16). The arch may be re-established, if additional bolts are installed along the edge of the intersection.

In recent years, more sophisticated numerical stress analyses were used. For example, Peng and Okubo (1978) analysed the stresses and displacements occurring around a four-way (+) panel intersection and a 3 way (Y and T)

intersection by using a three dimensional finite-element computer model. Their analyses were based on the data from 22 roof falls which occurred in an underground coal mine.

From the comparison of the dimensions of the observed roof falls with the vertical stress concentration contours over an intersection subjected to a vertical load equal to the overburden weight, they suggest the following roof fall criterion.

The induced vertical stress contour having a value of one-tenth of the in-situ vertical stress defines the boundaries of the arch zone above the intersection that has more or less loosened up. Figure 2.17 presents the results obtained by Peng et al (1978).

Based on the above results, they suggested that the roof stability could be improved by using the suspension effect of the bolting or by increasing the shear strength between strata. The suspension effect could be provided by larger and stronger bolts in the central region without altering the spacings of the bolts. To increase the shear strength, the design is more complicated because the coefficent of friction and the required anchorage capacity should be evaluated. Figure 2.18 and 2.19 present the general rock bolt pattern proposed for 4 way and different types of 3 way intersections to provide the suspension effect and to resist sliding between the strata.

Analytical Approach

Analytical study of stresses at intersections were carried out by a number of the researchers. Three-dimensional photoelasticity was used to analyse the stress distribution around openings by Nisada and Hando (1956), and Kaneshige et al (1964). These analyses were carried out with circular

holes bored transversally in circular bars loaded uniaxially in the direction perpendicular to the axes of holes. A very comprehensive series of tests were investigated to compute the stress distribution around horizontal, inclined and different level crossings. The peak tangential stress at the springline of intersection corner was evaluated. An example of their results is presented in Figure 2.20. The major conclusion derived from these studies was that, for intersection angles (α) larger than 45 degrees, the effect of intersection on the tangential stresses did not exceed two hole diameters from the corner but for smaller angles, the extent of the zone of influence reached to several hole diameters from the corner.

Hocking et al (1976, 1978) used a three dimensional boundary-element method to analyse a 3 way intersection of circular tunnels in an infinite, isotropic, homogeneous and linear elastic medium. He found that; under a unit vertical stress, the maxiumum stress concentration for "+" and "T" junctions were about twice that which would exist around a single tunnel under similar loading. Figure 2.21 summarizes the results obtained in their study.

More recently, Gercek (1982) used a three dimensional finite-element model to study the stresses and displacements of various forms of typical 3 way and 4 way intersections of room and pillar panels. The analysis was carried out to determine the stresses and deformations around a four-way intersection under a uniform horizontal and vertical in-situ stresses. In particular, the influence of the pillar size and the in-situ state of stress on the overall stability were modelled.

Typical graphs of the vertical displacement and vertical stresses obtained by Gercek are presented in Figure 2.22 and 2.23.

The following observations were highlighted by Gercek according to his initial models:

- The maximum vertical displacement above the intersection occurs when the horizontal in-situ stresses are minimum.

- The magnitude of the vertical displacement is also significantly affected by the elastic properties assigned to the rock types.

- The induced vertical stresses around an intersection is not much affected by the horizontal stress.

However, he recognized that the vertical stress alone is not adequate to describe the total stress state over an intersection. Accordingly a series of two dimensional models simulating the intersection openings were analyses.

The author described the state of stress over an intersection with a safety factor defined as the ratio of the strength of the rock and the stress induced. The m and s parameters of Hoek and Brown (1980) criterion were used to calculate the strength of the rock mass.

Strength parameters used were based on RMR (Rock mass rating) ranging from 65 to 85 according to the geomechanics classification. These analyses indicated that shear overstress would occur at the roof of the intersection and that the safety factor in the roof of the intersection would decrease as the horizontal in-situ stresses increases.

Gercek (1982) also compared the stabilities of "T" and "+" intersections developed in room-and-pillar coal mines. The model used was an elastic model consisting of layers of

sandstone, shale, interbedded coal and shale and a seam of coal. An insitu stress ratio of 0.5 ($k = \sigma h/\sigma v$) was assumed.

The principal conclusions from the analyses are summarized below.

- The maximum vertical convergence of "T" intersection occurs not at the center but a point close to the entrance of the branch entry and the roof convergence adjacent to the rib located against the branch is not affected.

- For an isolated intersection, the roof of the entry is affected within a region equal to the width of the entry.

- For a "+" intersection the maximum convergence occurs in the center.

- The maximum stress occurs at the pillar corners and the magnitude of this stress is higher for a "+" than a "T" intersection under the same conditions.

It is necessary to note that, Gercek's results do not include the presence of discontinuities which could significantly affect the strength of the rock and hence, limit the applicability of the results obtained. In addition, the computed local safety factor employ only the elastic stresses and they cannot account for the progressive failure of the rock mass. For these reasons, the actual failed zone which would exist in a coal mine could be quite different from what was computed.

However, based on the theoretical analysis described Gercek derived some design suggestions from his investigations.

- Sharp intersections are regions of high stresses and if possible, they should be avoided.

- Three way intersections should be more stable than four way intersections.

- Support measures determined for single openings should be increased gradually as the opening intersection approaches an intersection.

- Gradual increase of support measures or the application of additional support should start at a point which is at least one tunnel diameter (one entry width) away from the entrance to the intersection (Figure 2.24).

Observational and Geomechanics Approach

Unal (1983) used a different approach for the analysis of coal mine roof behavior. He studied the stability of an intersection on the basis of the Geomechanics Classification, the field characteristics and the rock-load height. Unal derived an empirical formula from the comparison of the field observations in U.S.A. coal mines with the results of a boundary element program. His formula is presented below:

$$ht = \frac{(100 - RMR)}{100} B \qquad (2.1)$$

Where ht = uniform rock load height in the center of the intersection (m)

B = diagonal span of an intersection (m)

RMR = Geomechanics Classification of the rock mass

He compared the rock load estimated with the above formula over an intersection, with other empirical estimations of rock load. A good correlation was found for the U.S. coal mine but the assumption of the linear variation of the height (ht) as a function of roof span has not been statistically evaluated. Furthermore, the comparison of the Unal's formula with an analytical and numerical analysis for the cases where the Geomechanics Classification (RMR) ranging from 25 < RMR < 85 and the in-situ stress ratio ($k_o=\sigma h/\sigma v$) ranging from 0.3 to 1.5 indicated that:

- the rock load height envelope predicted from the empirical equation forms an upper limit to the failure-height envelope determined by boundary element analyses.

- The formula is not affected by the stress ratio.

Based on his study, Unal proposed support guidelines for coal mine intersections covering a wide range of rock mass classes and roadway spans. The guidelines include the specifications and patterns for mechanical and resin bolts in combination with planks and timber posts. The guidelines are based on the principal design assumption that the bolt length is equal to the half of the rock load height (ht) evaluated by his formula (equation 2.1). However, the rationale of this assumption was not fully discussed in his work and it was applied to rock bolt support used with and without timber posts at the corners of the intersections. Figure 2.25 shows an example of his proposed chart.

Since 1956, the U.S. Bureau of Mines (Panek, 1973) has undertaken several studies to analyse the stability of bedded roof of U.S. coal mines. More recently, Hanna and Haramy (1986, 1988) instrumented two different intersections in a room and pillar layout in the Illinois coal basin. The

instrumentation program included the determination of geotechnical data, in situ stresses, strata movement, and bolt load at the intersections. The principal observations in both mines are as follows:

- An increase in the vertical pressure and a decrease in horizontal pressure in the pillar occurred until the face was, advanced two crosscuts or approximately 46 m.

- The rock load seemed to be influenced by both local roof deformation and geology and by the orientation of the horizontal principal stresses.

- The measured deflection of the intact roof were close to the deflections predicted by plate theory. Based on analysis of the observations and results of the instruments, the failure mechanism of the roof strata at the intersection in both mines was attributed to: the high in-situ horizontal stress ($\sigma h > \sigma v$), the particular geological conditions and delamination of layers caused by gravity.

Three principal design criteria were also identified:

- Minimize the time delay between excavation and installation of support.

- Minimize the development and progression of shear, interlayer slip, and delaminated zones.

- Minimize the diagonal roof span parallel to the major principal horizontal stress.

The following measures are suggested to satisfy the above criteria:

- Roof bolts should be installed as soon as possible after the excavation to create a beam over the intersection and to inhibit the deformation movement.

- Install conventional truss bolts or intersection truss systems as required (Figure 2.26), these systems should be installed as part of the excavation/support. The concept of truss bolt was (Figure 2.27) specially developed for supporting intersections. This special "Intérsection Truss" allows the forces from each diagonal sections to interact thus providing a continuous support system.

- To minimize the progressive failure, install longer bolts or angled bolts in the critical area (the edge and corners) adjacent to the pillar. This procedure will permit the transmission of loads to the pillar abutment.

- To minimize the diagonal span, and keep the rounded corners to a minimum.

2.3 Design of Roadway Intersections in South Africa

In general roof support in South African coal mines utilizes grouted rock bolts, or shell anchored rock bolts. Passive support is seldom installed except for unusually poor ground associated with faults, dyke intrusions or burned coal. Because of the evolutionary nature of the support design, little has been published which documents the methods used, and much of the information reported here is gleaned from first-hand knowledge or based on discussion with engineers working in the industry.

2.3.1 Rock Conditions

The coal deposits of South Africa are found in the Karoo Formation, which spans a period of more than 100 million years from the late Carboniferous (290myBP)* to the early Jurrassic (190myBP)*. The economic seams are mined in the Transvaal and the Northern Orange Free state and also in the Northern Natal areas. In general, the mining of coal by underground methods is rarely carried out at depths of greater than 200 m, with the consequence that about 90% of all coal mined by underground techniques is mined by bord and pillar (room and pillar) methods. Bords are typically 5.0 to 6.5 m in width and pillar sizes depend upon the depth to the floor of the seam.

The general geology in the proximity of the seams comprises siltstones and shales with varying amounts of sandstone. The Karoo coal deposits are allochthonous (subjected to transport) in origin which often results in the floor or roof contacts of the seam being erosional, and the coal having a detrital content higher than that typical of the Northern Hemisphere Carboniferous coals. The geology of a typical seam consists of an immediate roof of shale or mudstone, overlain in turn by alternating siltstones and

* MYBP: Million years before Precambrian

sandstone, with some of the sandstones being very competent. A dominant feature of the geology away from the immediate seam, is a thick sill, consisting of finely crystalline dolerite exhibiting vertical jointing and minor horizontal joints.

Many of the seams mined in the Transvaal and Orange Free State are in excess of 4.5 m thickness; consequently the roof of some working levels consists of coal or thin coal overlain by mudstone or shales. On occasions, seams are mined where the top contact of the coal is erosional and overlain directly by a competent sandstone horizon.

Where failure occurs, the mode of failure is usually dependent upon the geology of the immediate roof or the geology of the coal seam. Bedded mudstone or shale in the immediate roof tends to fail by horizontal exfoliation of the bedding causing slab like collapse. In highly laminated material, the failure may migrate upwards to form an arched roof to the cavity, as the span of each successive layer is reduced by the support of the layer immediately beneath it.

The failure of more competent sandstone rich immediate roof rocks tend to be a more typically blocky failure, with blocks being defined by intersecting subvertical joints associated with horizontal bedding discontinuities. The size of such blocks is totally dependent upon the frequency or spacing of the bounding discontinuities. In some areas, notably the Vaal Basin coal mines, areas exist where the mode of roof failure is clearly influenced by the failure of the coal pillars. In these areas the coal tends to be weakened by the presence of a strong cleat which often shows evidence of slickensiding. When the cleat is adversely orientated with respect to the roadways, slabbing of the coal pillar results in an increased roof span and can lead to collapse of an intersection which was otherwise adequately supported.

Due to the relatively shallow nature of the coal seams in South Africa, it has proved difficult to carry out stress determinations by any method of strain relief. Observational evidence exists which has lead many people to suggest that the horizontal stress exceeds the vertical stress, and that the vertical stress itself is a function of the depth and the rock density.

2.3.2 Excavation Methods

Almost without exception, the roadway shape used in South Africa is conformable with the stratification, which results in "flat-top" roadways. As indicated in the previous section, about 90% of coal mined by underground methods is mined using bord and pillar techniques, with the consequence that a great many four-way intersections are produced every day. Wagner and Madden (1984) state that about two thirds of all coal produced in South Africa is produced by bord and pillar techniques, and the Mining Review of 1987 indicates that a total of about 177 million tons of coal was produced in that year.

A great many intersections are mined using drill and blast techniques. The face of the coal is undercut with a coal cutter to produce the second free face which is required by regulations, and the face is then charged and blasted using permitted explosives. Most mines position the cut towards the bottom of the face, but a few use a cut towards the midline of the face. A method which has been used in areas of very weak roof, such as the Vaal Basin, is to utilize the cut to define the roof line. This precludes any damage being caused to the roof by the migration of blast gases from the top holes. However, unless the equipment is adequately modified for the purpose the final roof line, it can take on a stepped profile which is often a more difficult situation to support than is the blast damaged roof. The general method of forming an intersection by conventional blasting techniques is depicted in Figure

2.28. The cut-throughs or plists are turned off to either the left or the right in all roads across the section, and mined until they hole through into the adjacent roadway. Thus, each intersection is formed by one split mining away on side, and the opposite split holing into it from the other side.

Due to the fact that many collieries in South Africa operate a trackless mining system, many intersections are three and four-way right angled crossings. With the gradual increase in the use of machine cutting of coal, and the introduction of in-seam tracked transport of men and equipment, there is an increase in the number of acute angled intersections being formed. In other areas, the use of machine cutting has led to the introduction of diamond shaped pillars to reduce the angle at which splits are turned. Where machine cutting has been introduced the machines utilized are mostly continuous miners which cut a flat roofed profile, rather than heading machines which are more able to produce roadways with an arched profile.

2.3.3 Support Type and Practice

In a few of the older established Collieries in South Africa, support is still achieved by the use of passive timber and steel sets. However, over the past five to seven years, support technology has improved with the almost universal application of active bolted rock reinforcement techniques. In some areas of good competent roof, the bord width mined can be up to 5.5 or 6.0 m with no support installed. In such areas however, the increased roof span at intersections is such that roof bolting is often required. At the other end of the spectrum, some roof strata is very weak and friable or subject to sub-aerial weathering which requires the use of an intensive bolting pattern and other surface restraints such as wire mesh or steel straps.

Where passive systems are still utilized, the general philosophy in dealing with poor ground conditions is to reduce the roadway widths to a minimum in order that the roof beam will be stronger and to reduce the size and costs of heavier sets. However, such an approach is not appropriate to longwall extraction where the size of the roadways is likely to be optimized.

Over the years that roof bolting has been practiced in South Africa, a great range of bolt sizes and variations in grout have been tried, as well as the "original" shell anchored rock bolt. The main categories in general use are briefly described below:

i) Shell Anchored Bolts.
These require a hole of about 46 mm diameter to accommodate the anchor, and utilize a bolt of 12 mm to 15 mm diameter. The lengths available range between about 1.0 m and 2.4 m. Most installations utilize some form of surface washer or plate of between 100 mm and 150 mm diameter or side length, some of which are formed into a dome. This type of bolt is rapidly loosing favour, except in specific locations, because of the need to re-tighten after installation and the progressive loss in tension which occurs when anchored in "weaker" rock types.

ii) Point Anchor Resin Bolts
This type of bolt is the direct successor to the shell anchored bolt except that it utilizes a resin capsule as the point anchor. The hole size required is of the order of 25 mm to 28 mm in diameter and bolt lengths range between about 1.0 m and 3.4 m, although typical lengths are between about 1.5 m and 2.5 m. The diameter of the bolt is usually 12 mm but 16 mm bolts are also used. A surface or face plate

of similar size to that used with the shell anchored bolt is typical. There are two major limitations to this type of installation. Firstly, the drilled hole length is critical because an overdrilled hole will result in the bolt being anchored in only part of the resin capsule, with the remainder of the resin filling up the overdrilled zone. Secondly, this type of bolt must be tensioned when installed. If the resin capsule has not fully cured at the time of tensioning then the bolt may creep out of the resin. The situation can be made even worse if the resin is disturbed as this can breakdown the chemical curing process rendering the installation useless.

iii) Full Column Resin Bolts

As the name suggests, these bolts are installed fully bonded to the rock over their entire length. The hole diameter is usually 25 mm to 28 mm and the diameter of the bolt is either 12 mm or 16 mm. In order to reduce the unit costs, effort has been directed towards developing equipment capable of drilling 22 mm diameter holes to accommodate 12 mm diameter bolts. Although face plates or washers are fitted, the bolts are not usually tensioned. However, one system which was tried involved the insertion of a fast setting resin capsule at the end of the hole, and slow setting capsules in the remainder. After the bolt had been spun to mix the resin, tension was applied after the fast capsule had set but prior to the setting of the slow capsule. Because of the natural range of curing and setting times, and the sensitivity of the curing to temperature; in practice the technique was difficult to control.

iv) Special Supports

A number of proprietary supports have been tested in South Africa, such as the Split Set manufactured by Ingersoll-Rand. Although some successes were noted, in general such special supports are too expensive to justify their common usage.

In some instances pumped grouts, (resins and cements) are used for full column grouted support systems. Generally, the methods adopted by individual collieries are dependent upon the equipment available.

The timing of the installation of the support results from the need to work within the regulations yet optimize the mining cycle times. Many drill and blast bord and pillar mining sections utilize roof bolting machines rather than any hand held equipment. The mining cycle is arranged such that the roof bolter goes to the face to support the newly exposed roof as soon as the mucking cycle is complete. Where machine mining is used, especially in the development of main and trunk roadways, the machine does not move between faces as often, and so the roof bolter is able to gain access to the face less frequently. To counter this problem and to be able to install support close to the face (and ahead of the operator's cab) attempts have been made to mount roof bolting rigs onto the continuous miners.

2.3.4 Design Procedure

Unfortunately, nothing relating to the design of intersection supports appears to have been published in the literature nor in the South African Chamber of Mines reports, describing the methodology used in the design of roof support systems. In many cases the support pattern in use has evolved from the use of shell anchor bolts. Little, if any, formal design procedures are used, and

supports are lengthened or the pattern densified in response to failures.

In new mining areas where the support is to be designed, the geology of the immediate roof will be studied by drilling short cored holes and by inspecting areas of roof fall. The initial determination of the bolt lengths required will be based primarily upon the detailed study of the roof strata. The factor which tends to influence the spacing of the bolts is the spacing of subvertical or cross-cutting discontinuities coupled with the span of the roof. Where bolts are obviously required to support blocky roof formed by intersecting discontinuities, then the analysis is based upon the premise that each bolt is capable of supporting 10 tons. A bearing capacity of 10 tons has been accepted following early work by the South African Chamber of Mines which required that each component of a shell anchored bolt was required to be sufficiently strong to ensure that the total bolt installation would not fail under an applied tensile load of 10 tons. Since then 10 tons has become the accepted standard (for pull out tests) which bolts are expected to attain. In some instances this value has also been used as a contract requirement and a measure of quality assurance.

Once the basic support design has been selected then the proposals are submitted to the Inspector of Mines for approval as support rules. The Inspector of Mines use, as a guideline, the support systems installed by other operators in similar geographic and geological conditions. Once the approval is obtained, some collieries then embark on a programme of instrumentation in order to validate the support design or to obtain evidence and data to persuade the Inspector of Mines to sanction changes. Thus, on many occasions, the original design tends to over support the roof. In bedded roof, the instrumentation will set out to

determine the amount of roof sag or deflection of the beam, and to ascertain where any bed separation is occurring. From these results it will be possible to determine whether longer bolts, a denser pattern, or the use of full column resin grouted bolts are indicated.

2.3.5 Design Implementation

When conventional roof bolting is practiced, the installation sequence is based upon the need to integrate the support phase into the full mining cycle. In a drill and blast roadway, the support will be installed to within about 1 m of the face prior to the blast. After the blast the total unsupported span will be approximately 3.5 m. Face mucking equipment is capable of control by an operator standing beneath previously supported roof. Once the face is cleaned, the roof bolting crew moves in to scale and support. In South Africa most rock bolts are installed using bolting machines rather than hand-held equipment. The roof bolting machine is a fully integrated rig which:

a) applies the temporary support
b) drills the bolt hole
c) inserts the grout capsule or pumps the grout
d) inserts and spins the bolt to mix the grout capsule
e) applies tension to the bolt by tightening the nut on the bearing plate.

The sequence of support installation is thus a function of the capability of the machines in use.

Modifications to the support patterns are required by regulations to cope with variations in ground conditions. Generally these modifications take the form of extra bolts installed in order to support joints or other discontinuities, and to support brows in the roof caused by

changes in the height of the excavation. These modifications are generally made by the mining crews in response to localized ground changes. Of more significance are the modifications which are required as a result of major ground changes. In South Africa, faults and folds are generally not of major importance, but the intrusion of dolerite bodies (both dykes and sills) are of major significance as they usually result in the formation of burned and slickensided coal. Also of importance, from a support viewpoint, is the development of slickensided joints as these often lead to severe instability in the roof and can be difficult to support especially if they are of sub-vertical orientation.

The general requirement for both of these ground types is for some form of surface restraint, often in the coal rib-sides and in the roof. The methods of support used include:

a) perforated straps
b) wire mesh, both diamond fence and weld mesh
c) RSJs of varying sizes
d) Shotcrete or gunite

The surface restraints are generally used in association with denser patterns of roof or sidewall bolting.

The difficulties of installation are usually associated with the limitations of the equipment available. Some of the typical problems which re-occur can be listed as:

a) Inability to drill small diameter holes in hard sandstone.

b) Oversized hole diameters causing incomplete grouting.

c) Time expired resin or resin capsules damaged during transport.

d) Underdrilling of hole length causing bolts to stand proud from roof. This can be due to the inability of the machine to drill in areas of high roof.

e) Overlong holes causing resin capsules to be lost at the end of the hole.

f) Resin damaged by overmixing or by attempts to tension the bolt too soon.

Many of these problems are simple to rectify once they are identified, but in the interim can lead to poorly or even unsupported roof.

2.4 Design of Roadway Intersectiions in Australia

In this study, the roadway intersection support for Tahmoor Mine was used as the reference for rock conditions and roadway support practices generally adopted in Australia.

2.4.1 Rock Condition

The Tahmoor Mine is located in the southern Sydney Basin 80 km south-west of Sydney and mines The Bull's Coal at depths of approximately 420 m. The roof strata at this mine comprises an interbedded sequence of laminates, siltstones, shales and fine sandstones which vary in thickness from 3 m to 10 m below the more massive Triassic sandstone. High horizontal in situ stress has caused significant shear failure within the interbedded immediate roof strata. The in situ stresses measured at the sites are:

$\sigma 1$ (horizontal) = 18.0 - 21.3 MPa
$\sigma 2$ (horizontal) = 12.3 - 16.0 MPa
$\sigma 3$ (vertical) = 9.8 - 16.0 MPa

The variability of the principal stress ($\sigma 1$) often causes a range of roof conditions.

2.4.2 Excavation Method

In Australian practice, the method of multiple entry retreat longwall mining is used. Bord and pillar mining (room and pillar) is also practiced but its relative importance is anticipated to decrease (Sleerman 1986) due to high productivity potential of the longwall technique and the increase in average mining depth.

Typical roadways in Australian coal mines have been 5.5 m wide. However, current trends are to implement narrower headings of 4.2 m to 4.7 m to improve roof stability and reduce support requirements. Conventional road headers are generally used to develop the roadways.

2.4.3 Support Type and Practice

Over the past twenty years, considerable improvement has taken place in the roof support for roadways in Australian coal mines. The salient points relating to the evolutions of roadway support methodology are:

- Timber cross support has virtually been phased out, being replaced by systems of steel strapping and resin anchored roof bolts.
- a decline in the use of timber props
- more extensive use of rib reinforcement techniques
- the increased use of rockbolts of different types including cold rolled threaded bolts and high tensile steel bolts
- the installation of supports as close to the face as possible
- the use of cable bolts for reinforcement of strata above normal bolting horizons
- development of monolithic cementitious support as a replacement for timber cribs

A description of the support types used for roadway support are given below.

Roof Bolts

Roof bolts are by far the most widely used form of underground support in the coal mines. 24 mm diameter deformed bars are the most common roof bolts. Resin anchors are the most frequently used grouting medium.

Frictional anchors such as split set and Swellex bolts have been field tested but these anchors have not been widely used.

Cross Support

Cross support has been utilized in one form or another since underground coal mining commenced in Australia. Traditionally, these supports comprise timber in the form of baulks or half round bars held to the roof by timber legs. Due to the increasing costs, the potential for the timber to deteriorate and the limited load carrying capacity in timber, steel beams are more often used for cross supports held to the roof by timber props, steel legs, roof bolts or combinations of such.

With the increasing confidence in the use of roof bolts for strata control, cross support usage has dropped significantly in recent years. The application is now primarily limited to areas having unstable roof conditions caused by bed separation which is higher than the roof bolts anchor horizon, localized stress zone and/or other adverse geological conditions.

Steel Arches

Steel arches were used in underground coal mines when:

- long term life of an opening is required,
- surrounding strata is highly unstable, and
- loading is too severe for other means of support.

Cable Bolts

In certain circumstances, cable bolts have been utilized as either active or passive support to reinforce an existing roof bolt pattern in unstable ground conditions. The application of cable bolts for ground support was introduced into Australia in the late 1960's. Cement grouted cable bolts were initially used. The current development is to use high tensile wire cables with resin anchors. The bolting pattern, as well as the drilling and installation technique are still in the development stage.

The roof support systems used in the Tahmoor mine (Richmond et al 1986) provides a profile of the application of various support types in the underground coal mines.

There have been three main generations of roof support used since mining began at Tahmoor. Each system had variations to cater for different rock conditions.

The support systems as reported by Richmond et al are summarized below.

a) First Generation Support: the roof support used at the beginning of mining in Tahmoor. It consisted of W straps (4.85 m) at 1 m spacing containing 5 x 2.1 m mild steel bolts point anchored with one 600 mm chemical cartridge. Sheets of mesh (3.0 x 2.5 m) were fitted prior to the W straps. Minor trials were made with 1.5 m bolts and wooden half round bars.

   This support system was effective for the good mining direction in the early development but proved unsuitable in the bad direction because

a. the area of roof exposed in each cut out prior to bolting needed to be large enough to fit the sheets of mesh which allowed too much ply separation and shearing, and

b. roadways were commonly driven wider than the W-strap could cover allowing guttering to develop. Alternatively rib deterioration and flaking exposed the roof beyond the coverage of roof support also allowing guttering.

b) Second Generation Support: for the purpose of adding strength to the support in bad conditions, or important roadways, RSJ's were used with the first generation support although the use of mesh diminished. The RSJ's were held by 2 x 900 mm chemical anchored bolts and placed between every second W-strap. Roadway width was reduced to 4.7 m to improve the roof standup time.

c) Third Generation Support was subsequently developed to cater for side of the roadway at which guttering was expected. This support pattern is described by Figure 2.29 and consists of

a. 6 bolts/W-strap for normal conditions, usually in the good drivage direction,

b. 7 bolts/strap for a mining direction where an extra bolt is placed on the ribside expected to gutter (as determined from the stress direction), and

c. 8 bolts per strap in the more severe conditions where rib shearing and guttering occurs on both sides of the roadway.

Butterfly straps and bolts are used to support any smaller areas of roof not covered by the prescribed support pattern. Wooden props are not considered as a primary roof support and their present limited use will probably be eliminated in the near future.

The main objective of the third generation support system is to curtail guttering a long the roadway and at the corners of intersections. Roof sag can still occur but the roof is held sufficiently by this support. The success of this support system is based on where and how much to increase, or decrease, the number of bolts placed as the strata pressure and stress direction change. Figure 2.30 shows a typical support pattern, with roof conditions typically encountered under the third generation support system.

2.4.4 Design Procedure

No explicit design procedure has been published for the support of roadways and their intersections. The support system described in the previous sections were likely developed based on the experience of field implementation. Nevertheless, Richmond et al (1986) has outlined the factors which should be considered in the design of roadway support. These factors might not have been completely evaluated for the support design but they indicate that a systematic design procedure is being developed. A summary of the design criteria is given below:

- Roadway geometry including width and shapes
- In situ stress condition
- Strata material properties including slaking and swelling properties, roof deformation characteristics, influence of roof and coal structure and discontinuities.

- Purpose of roadway and equipment used (such as roof bolter)
- Type of roadway failure mechanisms (e.g. span sagging, roof spalling, minor and major kinematic failure, shear and tensile failure and rib side buckling).

2.5 Summary

A review has been undertaken for roof supports for main roadways and roadway intersections in underground coal mines in the United Kingdom, United States, South Africa and Australia. A summary of the rock conditions, the excavation methods used, the roof support practice and the design procedures is given in Table 2.4. The data obtained from the coal mines in the Sydney coalfield are also included in the same table for comparison purposes.

Data regarding roadway intersection design in other European coal mines are proprietary information which are not readily available in published literature. Based on the limited data from published literature on other aspects of coal mining in Europe, the typical support system and design methodology are similar to those used in the U.K. In general, passive supports (steel sets) are used with limited roof bolting. In German Coal Mines, rock bolts have been used in combination with wire mesh in gate roads (Reuther et al 1986). However, the long tradition with steel set supports has limited wider application of rock bolting.

Similarily, very limited data have been published in roadway support design used in underground coal mines in China. Limited literature on the support of weak strata such as coal measures indicates that the type of support used ranges from complete steel arches in circular and horseshoe shapes to the use of rock bolts and shotcrete depending on the conditions encountered. U-shape steel

supports are often used as components for yielding arches under severe ground conditions.

In summary, it has been noted that the tradition of using steel supports still dominates coal mining in the U.K., Europe and Canada. The roof support system in the United States on the other hand is exclusively rock bolting. In South Africa and Australia, the current trend is to use rock bolts in the place of passive supports such as timber posts and steel arches. Steel sets are used only in very poor ground conditions.

In reviewing the support system for main roadway intersections, one can see the slow evolution of the support design and practice. The noteable points are presented as follows:

- There is a tendency for coal mines to use an active support system such as rock bolts to reinforce areas which have traditionally been supported by steel arches or timber props.

- Rock bolts are often used with supplementary support elements such as straps to bridge between the rock bolts.

- There is greater awareness of the need for monitoring the behaviour of the roof strata so as to assess the performance of the support system installed. The United States is very active in the research work related to the roof stability and support performance.

- Mines in which steel sets have traditionally been used are considering the use of roof bolting in areas where such support would provide a viable alternative to steel sets. Countries such as the U.K. and Canada are starting to implement

experimental programs to study the viability of roof bolting.

- The geotechnical design of either active or passive support systems for roadway intersections is still very much based on empirical or semi-empirical methods. Analytical methods such as numerical modelling are used only in a few occasions but are generally limited to research oriented studies. Peng et al (1978) developed guidelines for roof support in U.S. Coal mines based on numerical modelling calibrated by the depth of roof falls. Unal (1983) proposed a series of roof bolting patterns with two different type of bolts for a range of roof conditions according to a rock mass classification scheme. However, more field data are required to correlate the support performance to the recommended support requirements.

- Guidelines exist in the U.S. Code of Federal Regulations, Title 30 for selection of rock bolting support for coal-mine roof.

TABLE 2.1

AN EXAMPLE OF COLLIERY SUPPORT RULES (UNITED KINGDOM)

1. The minimum ground shall be exposed to erect the first junction cross-girder in proximity to the last heading support. Legs to be set on foot-blocks or similar base as directed by the management.

2. Each successive girder shall be raised into position using an approved lifting method or device.

3. A minimum of two horse-head girders shall be used at all times in order that the cross-girder being set is initially maneouvred into position, assisted by at least two of the horse-head girders.

4. An approved form of temporary support of exposed strata shall be used until permanent support is erected or the rock face advanced.

5. Depending upon the method of junction construction, one of the approved methods of temporarily supporting the cross-girders with steel legs shall be used.

6. All cross-girders shall be secured by at least four struts and at least three struts shall be used between temporary steel legs.

7. Carrier girders shall be raised into position by using an approved lifting device or method.

8. Approved packing material shall be placed between the steel surfaces of the carriers and cross-girders.

9. The permanent carrier stanchions shall be erected on a firm base, and an approved type of packing material placed between the steel bearing surface of the stanchion and the carrier girder.

10. Any cavity or exposed strata shall be packed or secured as the junction progresses.

11. The junction shall be completed by ensuring that no areas of exposed strata exist which have not been packed or dowelled where necessary to prevent possible danger from falling debris.

12. Permanent legs to be bolted to girders. Carriers to be bolted to stanchions. Stanchions to be bolted to floor.

13. Nothing in these rules shall prevent the setting of additional supports where necessary.

TABLE 2.2 Properties of Rock at a Illinois Basin Mine (110 m)

Stratigraphy	Thickness m (ft)	Cohesion (MPa)	Internal Angle (degree)	RQD (%)	Uniaxial Compressive Strength (MPa)	Elasticity Modulus (MPa)	Poisson's Ratio
Siltstone	-	5.93	27.4	100	19.31	6000	0.26
Sandstone	1 m (3 ft)	20.3	37.4	86	81.4	26000	0.26
Limestone	N.A.	N.A.	N.A.	100	N.A.	N.A.	N.A.
Shale	N.A.	N.A.	N.A.	0.0	N.A.	N.A.	N.A.
Brereton Limestone	0.6 to 2.5 m (2 to 10 ft)	41.8	17.4	84	113.8	29517	0.31
*Anna Shale	.30 to 1.8 m (1 to 5 ft)	N.A.	N.A.	0.0	N.A.	N.A.	N.A.
Herin No. 6 Coal	2.1 m (7 ft)	N.A.	N.A.	N.A.	20.0	2324	0.18 to 0.35
Mudstone	N.A.	N.A.	N.A.	57	N.A.	N.A.	N.A.
Limestone	N.A.	N.A.	N.A.	100	N.A.	N.A.	N.A.

Notes:

1. *The Anna shale consists of two distinct subunits a) hard, black, fissile, staley shale and b) poorly bedded, mottled, weak shale containing two persistent thin bands of phosphatic modules.

2. N.A. - not available from reference.

3. Ref: Hanna and Haramy 1985

TABLE 2.3 - Properties of Rock at a Illinois Basin Mine (293 m)

Stratigraphy	Thickness m (ft)	Cohesion (MPa)	Internal Angle (degree)	RQD (%)	Uniaxial Compressive Strength (MPa)	Elasticity Modulus (MPa)	Poisson's Ratio
Shale Siltstone	19 m (60 ft)	1.66	27.0		86.5	43931	N.A.
*Fossile Shale	N.A.	N.A.	N.A.		47 to 55	25724	N.A.
Coal	2 m (6 ft)	N.A.	N.A.		30	10758	N.A.
Free Clay and Shale	N.A.	N.A.	N.A.		64.7	35517	N.A.

Notes:

1. *The immediate roof rock is fine grained, dark to medium grey and contains abundant mica and carbonaceous debris. The roof is Drakensberg shale.

2. N.A. - not available from reference.

3. Ref: Hanna and Haramy 1986b

TABLE 2.4
Summary of Review of Roadway Intersection Design

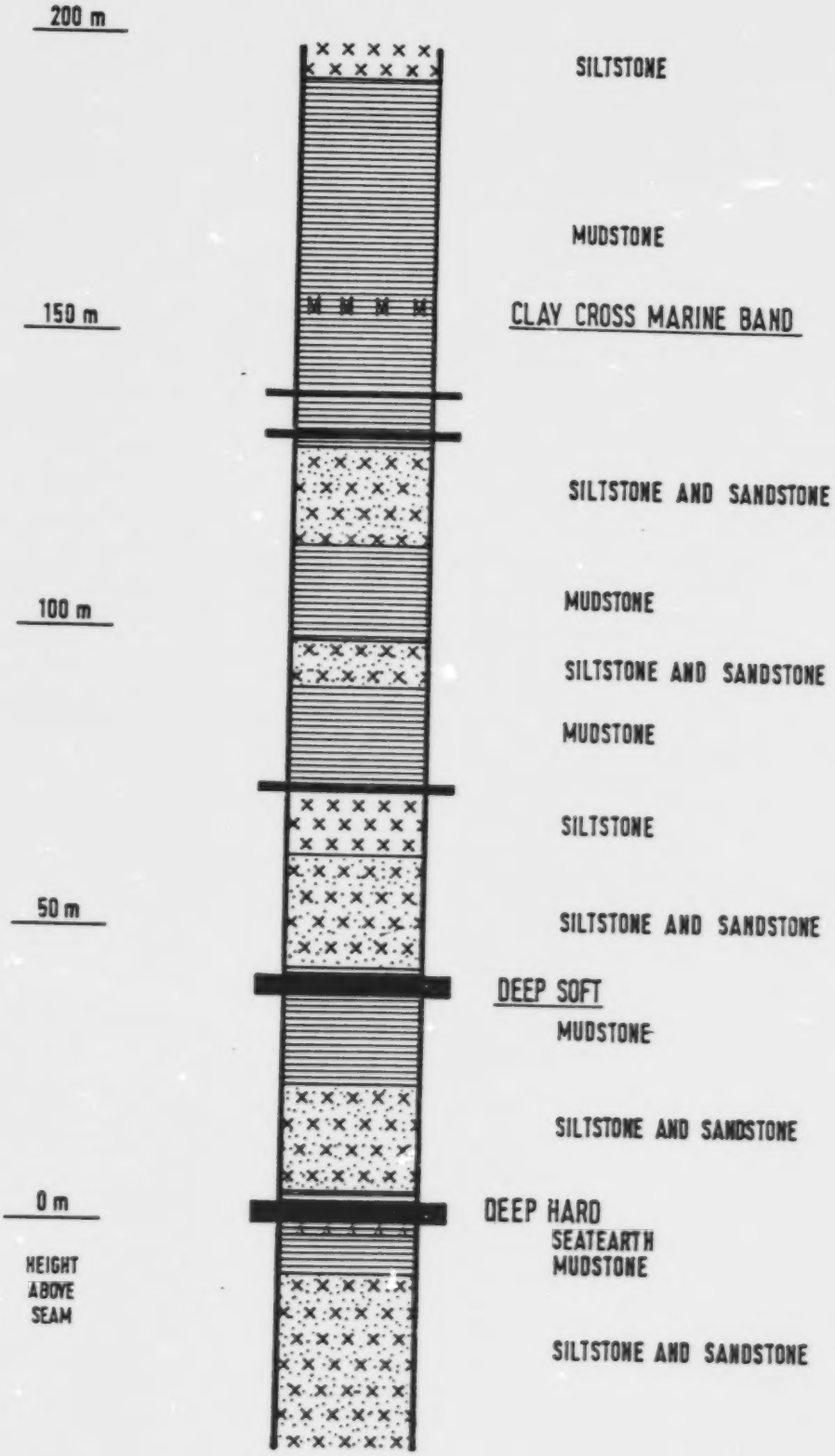
Country	Rock Condition	Excavation Method	Support Type	Design Procedure
United Kingdom	depth: 300 to 1000 m roof: -fine grained sandstone -siltstone -mudstone -coal Stress: K= σ h/σ v=2.0 σh anisotropic	-longwall mining -roadheaders	PASSIVE SUPPORT -steel arches (variable size) -steel sets -trapezoidal square work ACTIVE SUPPORT -resin bolts (20 to 25 mm diam) -roof bolts	-design of passive support is based on empirical rules and precedence -empirical and analytical approaches for active support -support legislation
United States	depth: 90 to 500 m roof: -horizontal bedding planes -shale -siltstones -sandstones -coal stress: $\sigma_h >> \sigma_v$ -principal stress E-W or NE-SW	-room pillar method (91%) -longwall (8%) -shortwall (1%) -continuous miner -tunnel boring machine (TBM) -boom-type roadheader	ACTIVE SUPPORT -mechanical anchor bolt -resin bolt -roof truss PASSIVE SUPPORT -cross beam -wooden post -cribs -crossbar -steel beam -wire mesh -shotcrete	-trial and error -experience -more and more based on the empirical, analytical and observational results obtained by different researchers -legislation for the support
Canada (CBDC)	depth: 90 to 700 m roof: -sandstone -more or less laminated siltstone -mudstone -coal stress: principal stress σ h > σ_v σ h anisotropic	-longwall -drill and blast -continuous miner	PASSIVE SUPPORT -cribs -post -pack -steel arches -"I" beams	-experience -capacity to deform -distributed load over the support

TABLE 2.4 (CONT'D)

Country	Rock Condition	Excavation Method	Support Type	Design Procedure
South Africa	depth: <200 m roof: -siltstones -shales (mudstones) -sandstones -coal with a detrital content stress: relatively low $\sigma h > \sigma v$	-bord and pillars (90%) -drill and blast to create the intersections with undercut	ACTIVE SUPPORT -grouted rock bolts (12to16mm dia.) (1.5to2.5m length) -split set PASSIVE SUPPORT (old mines) -timber -steel sets	-drilling short cored holes to analyse the roof geology -geology defined the initial bolt length -adjustment of the bolting pattern with the ground conditions -support pattern approved by the Inspector of Mines -program of instrumentation to approve the bolt pattern
Australia	depth: variable ≈400 m roof: interbedded laminates, siltstones, shales and fine sandstones stress: principal stress= σh $\sigma h > \sigma v$ σh anisotropic	-multiple entry retreat longwall mining -bord and pillar -conventional roadheaders	ACTIVE SUPPORT -steel strapping -resin bolts (24 mm dia.) -threaded bolts -high tensile steel bolts -cable bolts PASSIVE SUPPORT -timber cross -timber props -steel arches when the ground conditions are severe	-no explicit design procedure -field experience -design procedure is being developed

GEOLOGY ABOVE THE DEEP HARD SEAM
BRITISH COAL WESTERN AREA

FIG. 2.1



Date APRIL 21, 1988
Project 871-1369

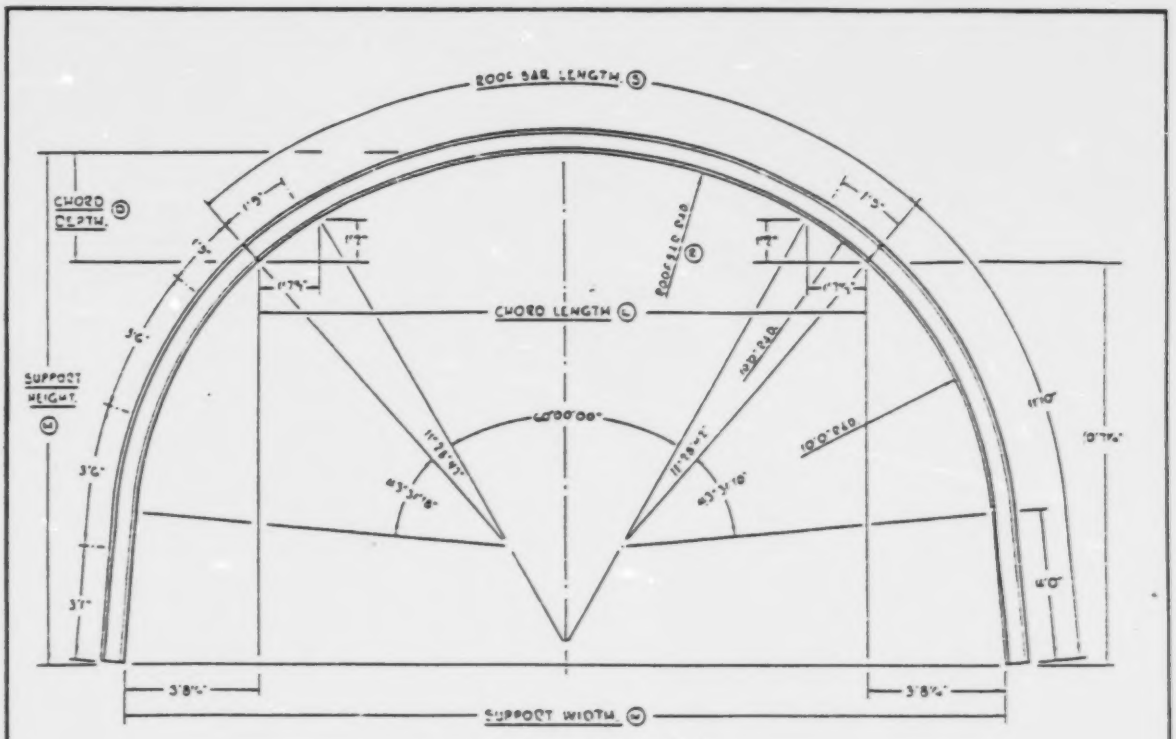
Golder Associates

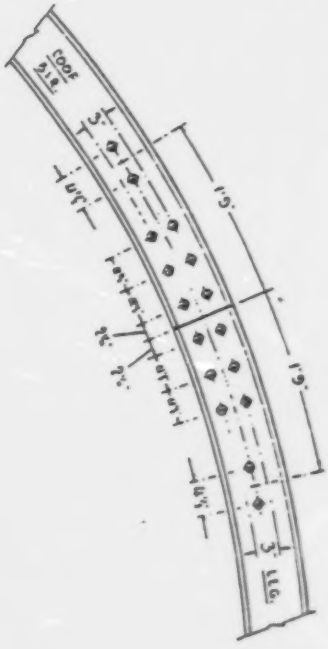
Drawn D.M.
Chkd F.M.

DATE MAY 88
REVISED BY
DR A 1031 1

FISHPLATED JUNCTION ARCHES

FIG. 2.2





Available for 12 hole or 4 hole fishplates.

For full range of Dimensions see accompanying Table.

Note: For all support sizes a common leg length is used.

Date APRIL 25, 1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd F. M.

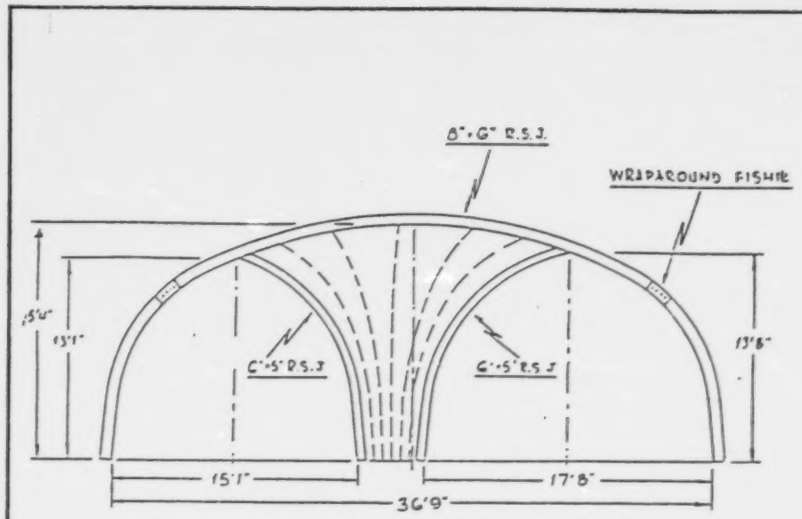
DRG No A 871037 002

REVIEWED BY

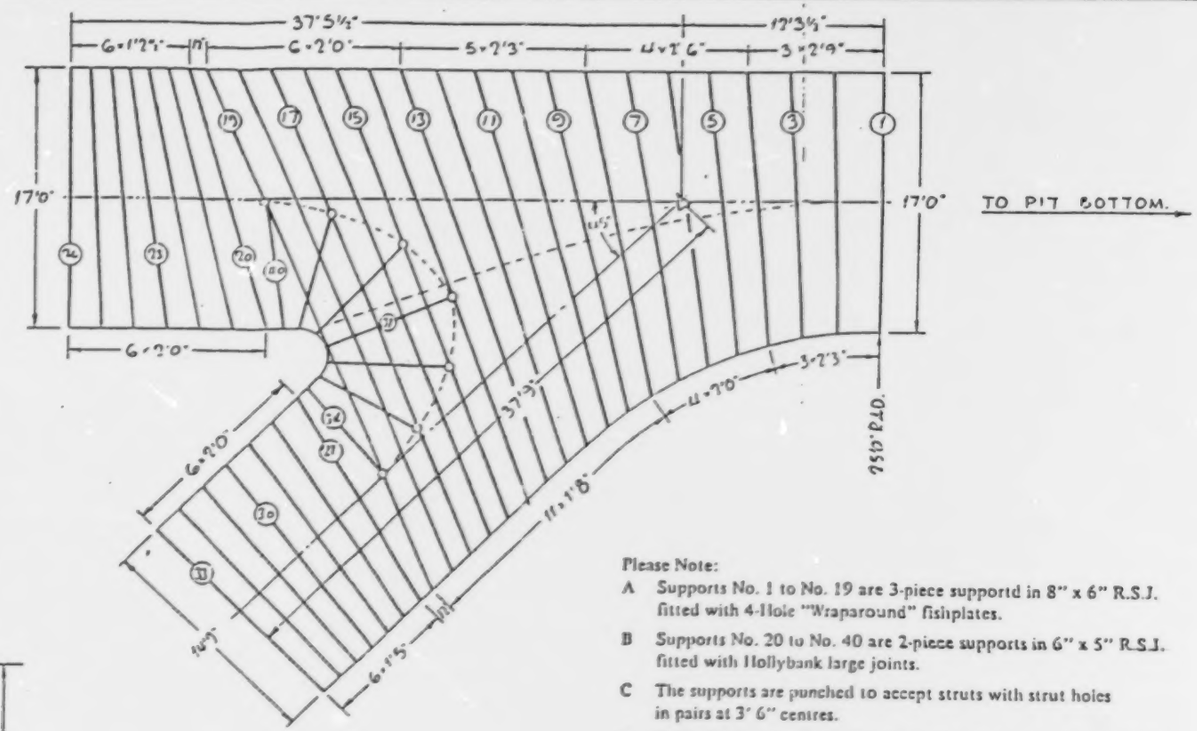
DAY

JUNCTION USING FISHPLATED SUPPORTS

FIG. 2.3



Support No. 19



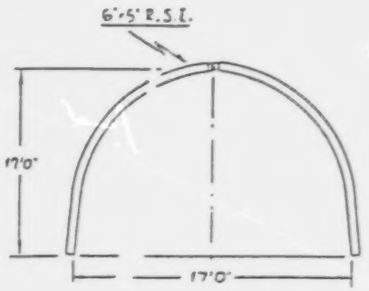
Please Note:

A Supports No. 1 to No. 19 are 3-piece supportd in 8" x 6" R.S.J. fitted with 4-Hole "Wraparound" fishplates.

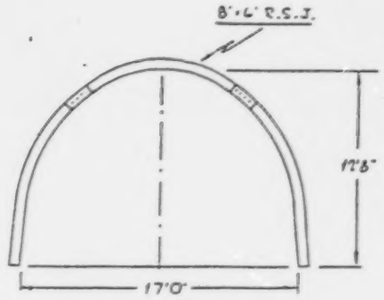
B Supports No. 20 to No. 40 are 2-piece supports in 6" x 5" R.S.J. fitted with Hollybank large joints.

C The supports are punched to accept struts with strut holes in pairs at 3' 6" centres.

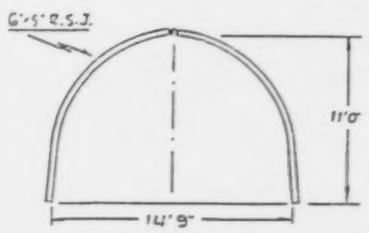
D The "Palmtree" supports No. 34 to No. 40 are extra and subject to an additional price (see formal quotation).



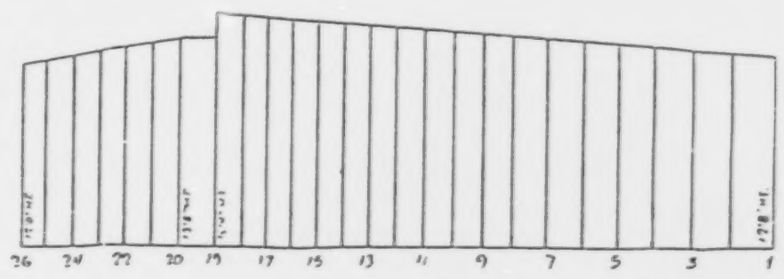
Support No. 26



Support No. 1



Support No. 33



Section on Support 1

Ref.: HOLLYBANK Roof Support Systems for Modern Mining

Date MAY 26, 1988

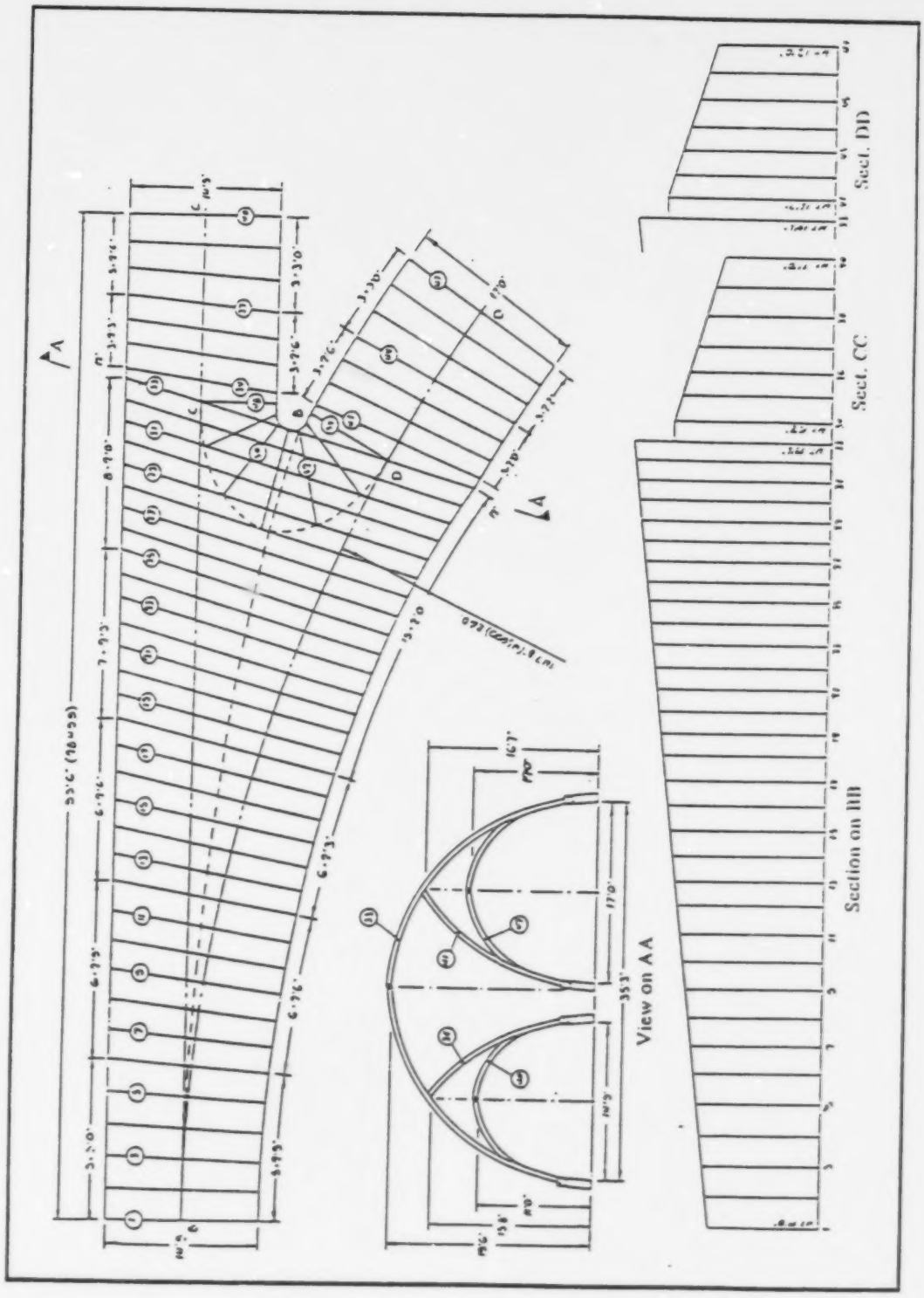
Project 871-1369

Golder Associates

Drawn D. M.

Chkd. F. M.

JUNCTION USING FLEXIBLE JOINTS AND YIELD BOXES | FIG. 2.4



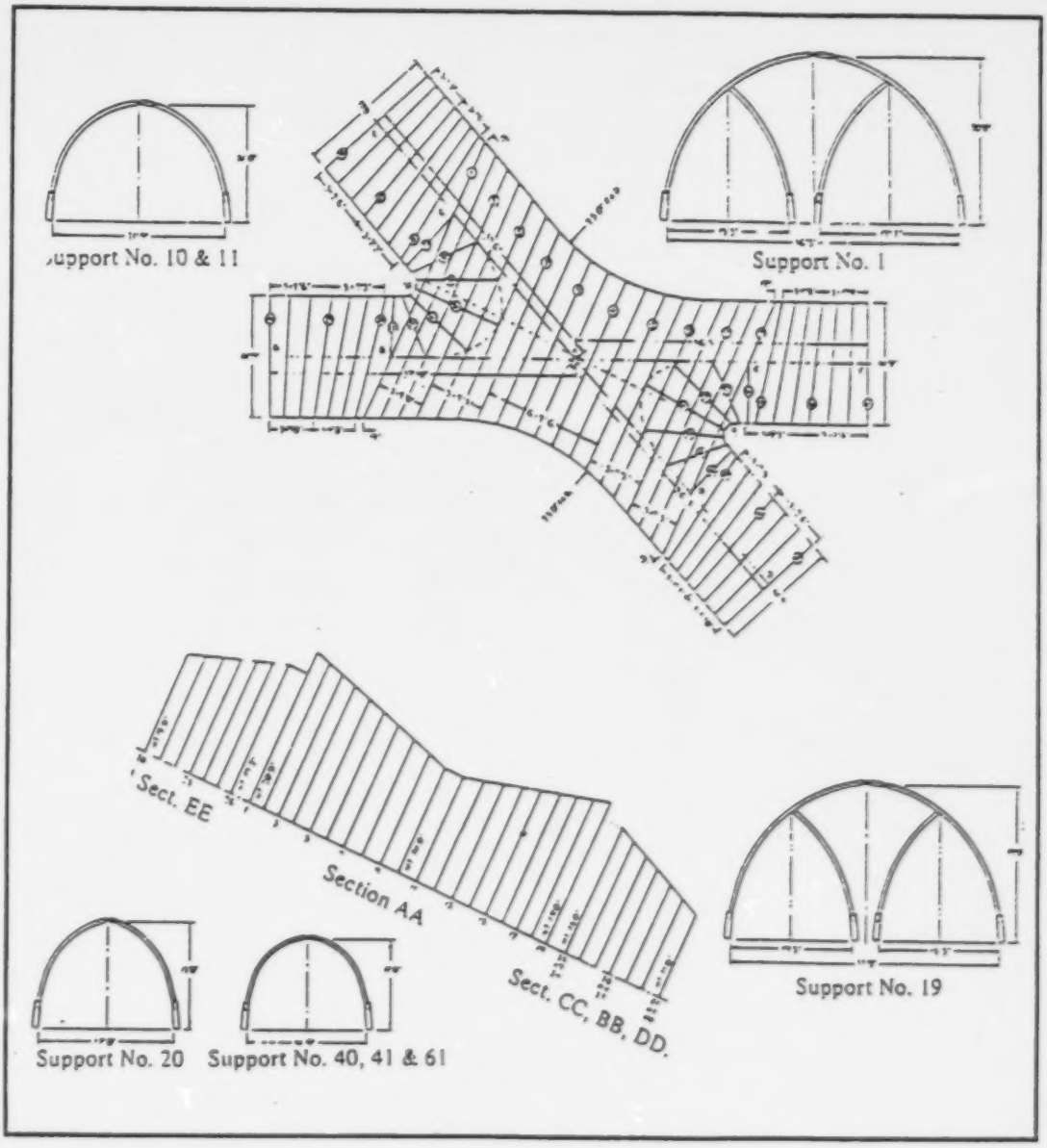
Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.M

TYPICAL 4-WAY JUNCTION

FIG. 2.5



Date MAY 26, 1988
Project 871-1369

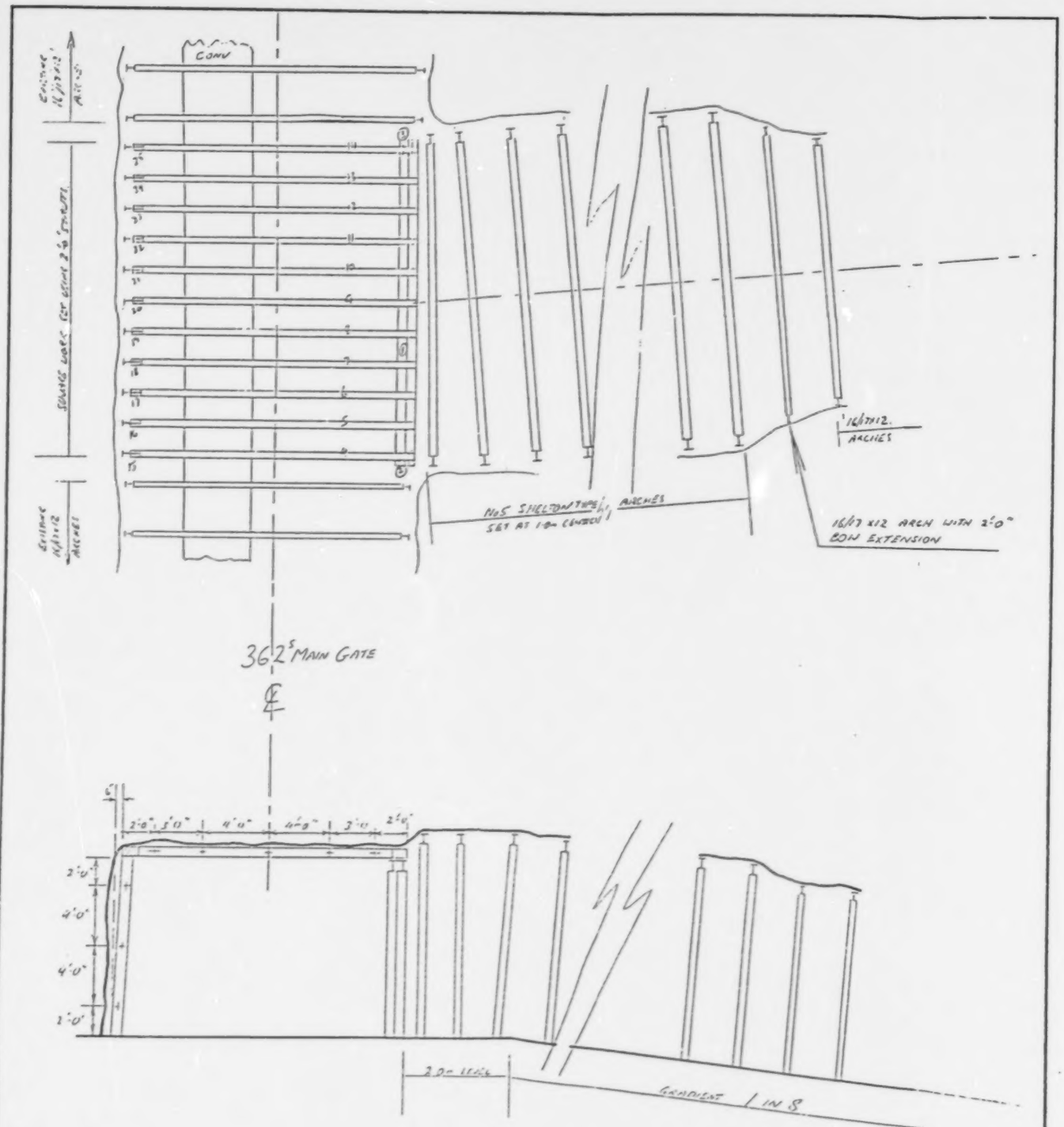
Golder Associates

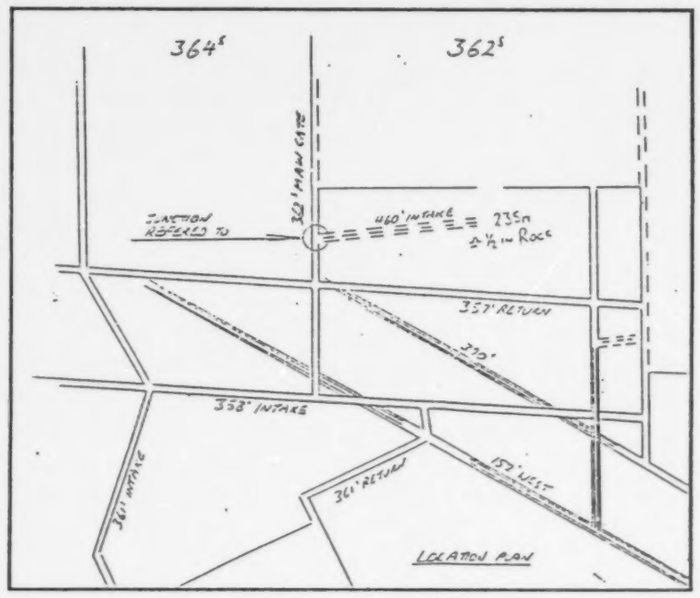
Drawn D. M.
Chkd E. W.

DRAWING A 103 05 REVIEWED BY DATE MAY

JUNCTION SUPPORT FABRICATED AT THE COLLIERY

FIG. 2.6

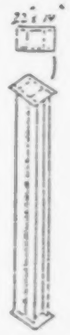




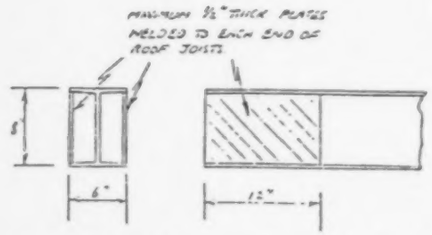
NB.- 1. CARRIERS TO BE ROOF BOLTED
2. TOPS OF STANCHIONS TO BE PAINTED
3 TIMBER BLOCKS TO BE PLACED BETWEEN STEELWORK
4 FOOT BLOCKS TO BE USED AT BASE OF STANCHIONS

COMPOUND CARRIER

10"x8" R.S.T



COMPOUND STANCHION 10"x8" R.S.T MINIMUM



METHOD OF STRENGTHENING ROOF JOISTS No: 4 TO 14

Date MAY 26, 1988
Project 871-1369

Golder Associates

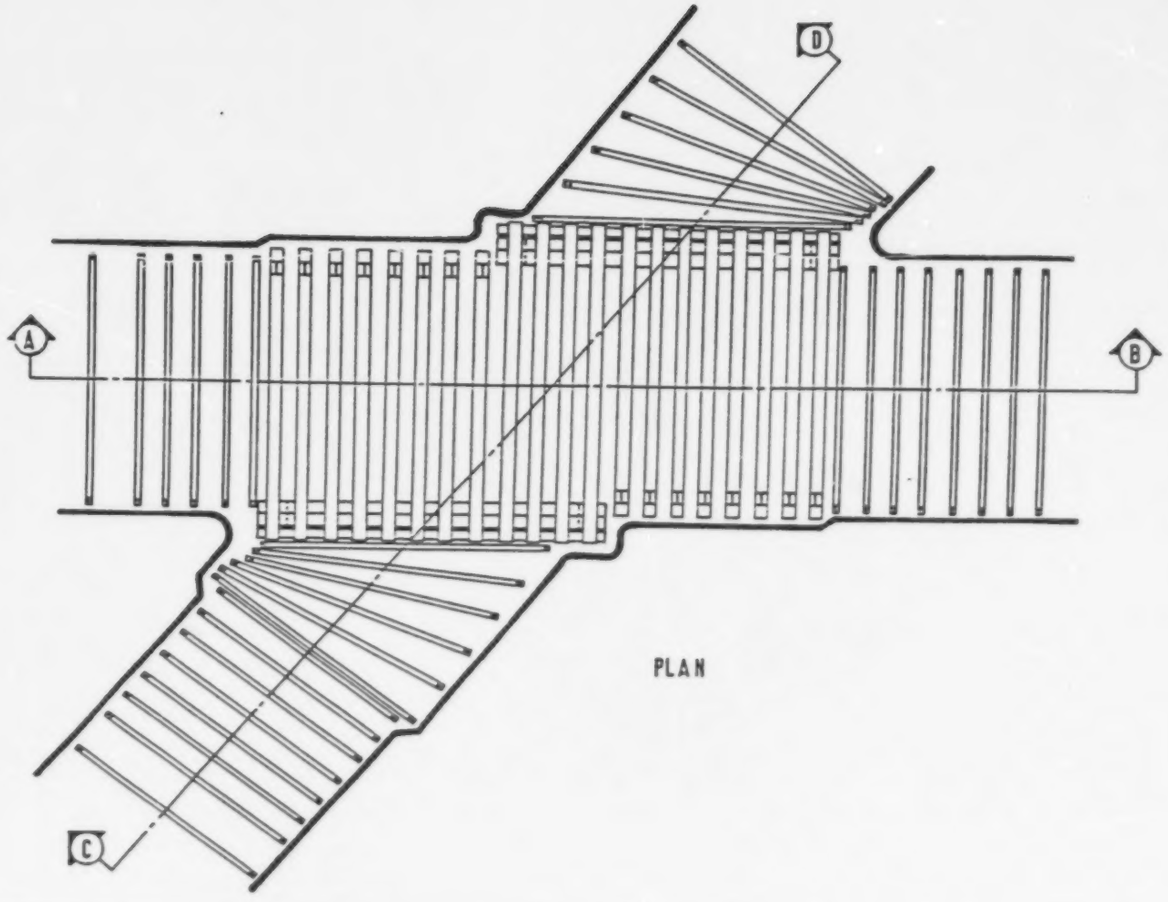
Drawn D.M.
Chkd F.M.

REVIEWED BY:

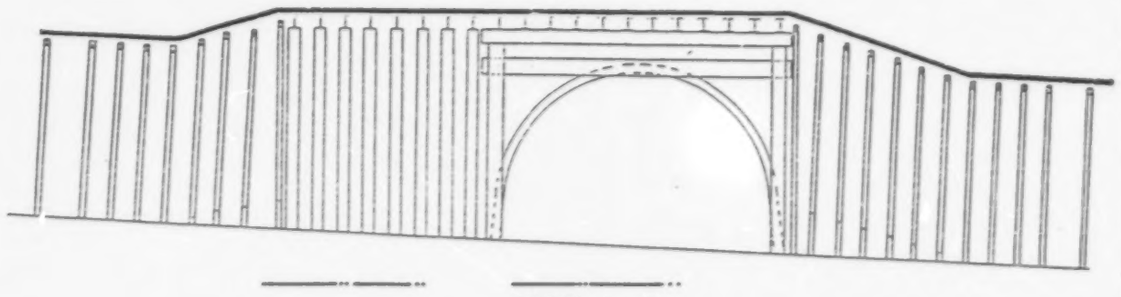
DRG. No. B 37 2 103 030

JUNCTION SUPPORT FABRICATED AT THE COLLIERY

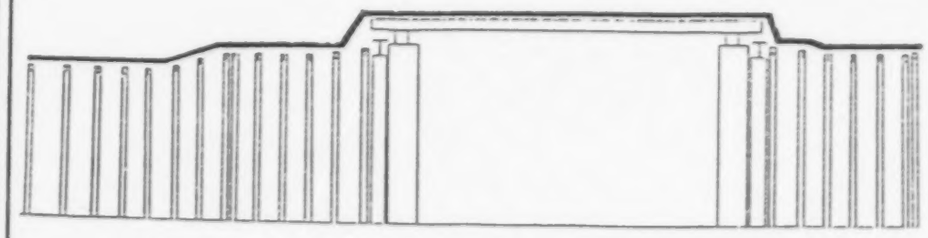
FIG 2.7



PLAN



SECTION A - B



SECTION C - D

SUPPORT RULES

1. The minimum ground shall be exposed to erect the first junction cross-girder in proximity to the last heading support. Legs to be set on foot-blocks or similar base as directed by the management.
2. Each successive girder shall be raised into position using an approved lifting method or device.
3. A minimum of two horse-head girders shall be used at all times in order that the cross-girder being set is initially manoeuvred into position, assisted by at least two of the horse-head girders.
4. An approved form of temporary support of exposed strata shall be used until permanent support is erected or the rock face advanced.
5. Depending upon the method of junction construction, one of the approved methods of temporarily supporting the cross-girders with steel legs shall be used
6. All cross-girders shall be secured by at least four struts and at least three struts shall be used between temporary steel legs.
7. Carrier girders shall be raised into position by using an approved lifting device or method.
8. Approved packing material shall be placed between the steel surfaces of the carriers and cross-girders
9. The permanent carrier stanchions shall be erected on a firm base, and an approved type of packing material placed between the steel bearing surface of the stanchion and the carrier girder.
10. Any cavity or exposed strata shall be packed or secured as the junction progresses.
11. The junction shall be completed by ensuring that no areas of exposed strata exist which have not been packed or dowelled where necessary to prevent possible danger from falling debris.
12. Nothing in these rules shall prevent the setting of additional supports where necessary
13. Permanent legs to be bolted to girders. Carriers to be bolted to stanchions. Stanchions to be bolted to floor.

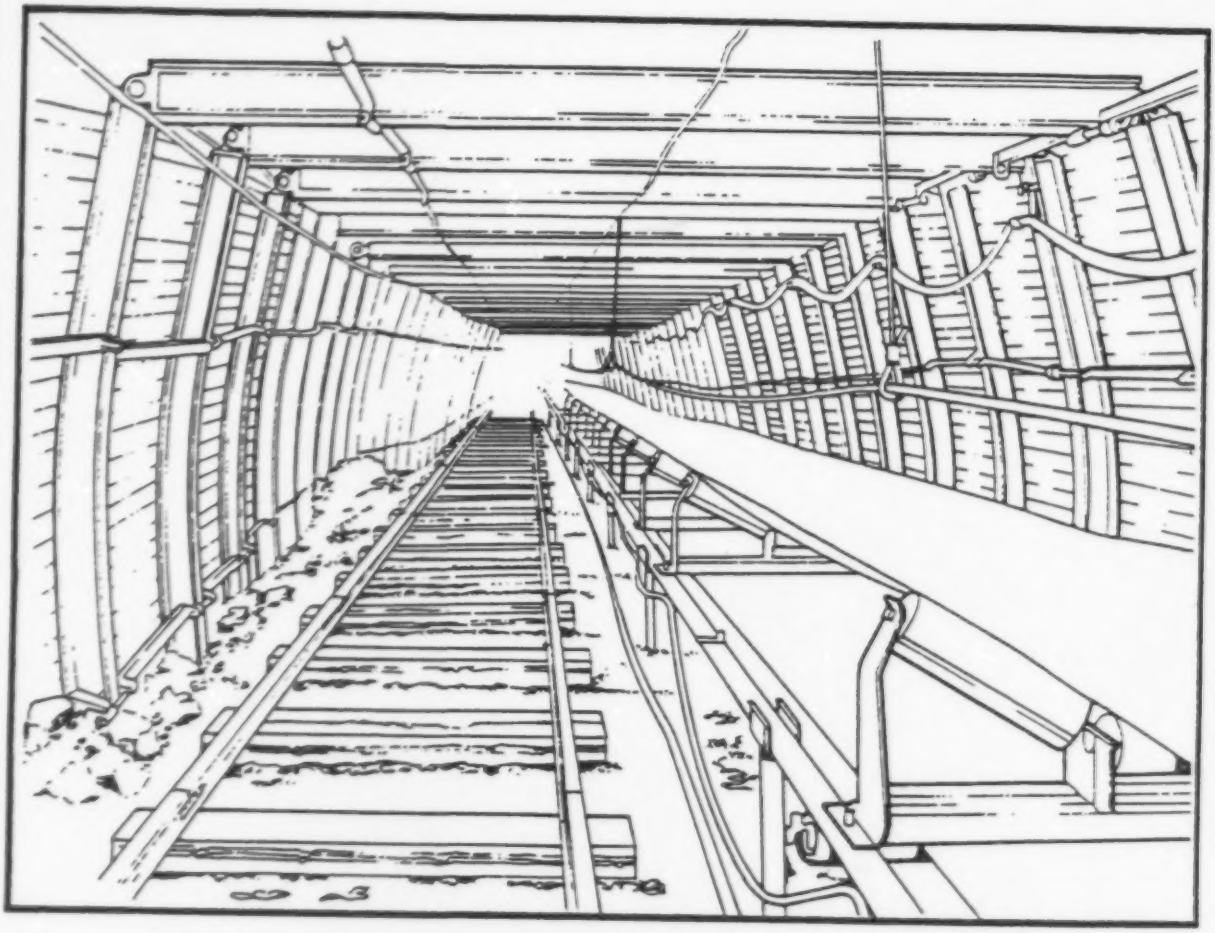
Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.M.

EXAMPLE OF TRAPEZOIDAL SUPPORT

FIG. 2.8



Date MAY 26, 1988

Project 871-1369

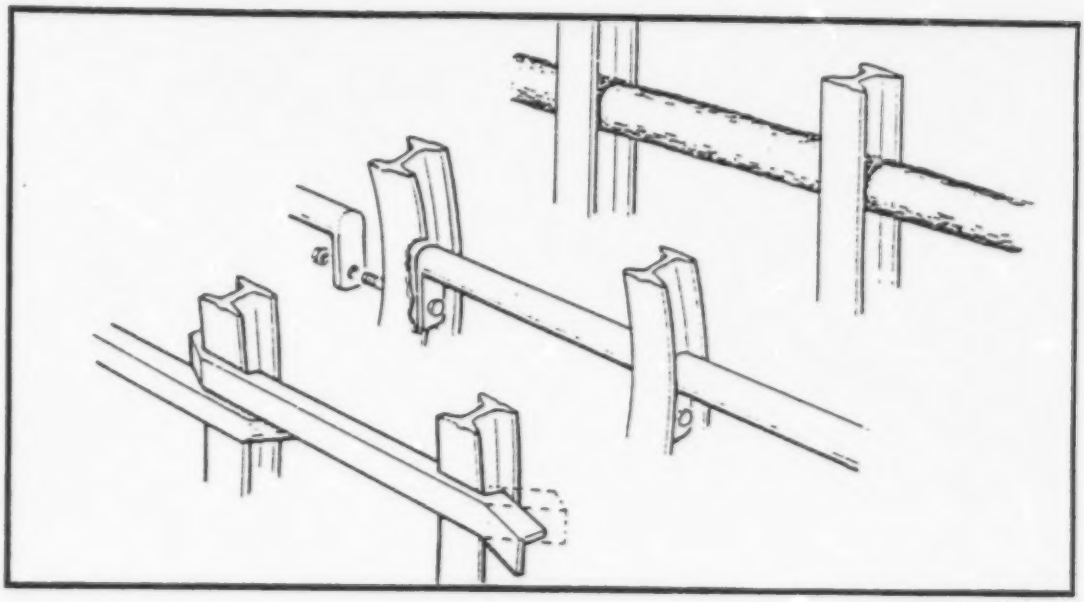
Golder Associates

Drawn D.M.

Chkd F.M.

EXAMPLE OF TYPES OF STRUT

FIG. 2.9



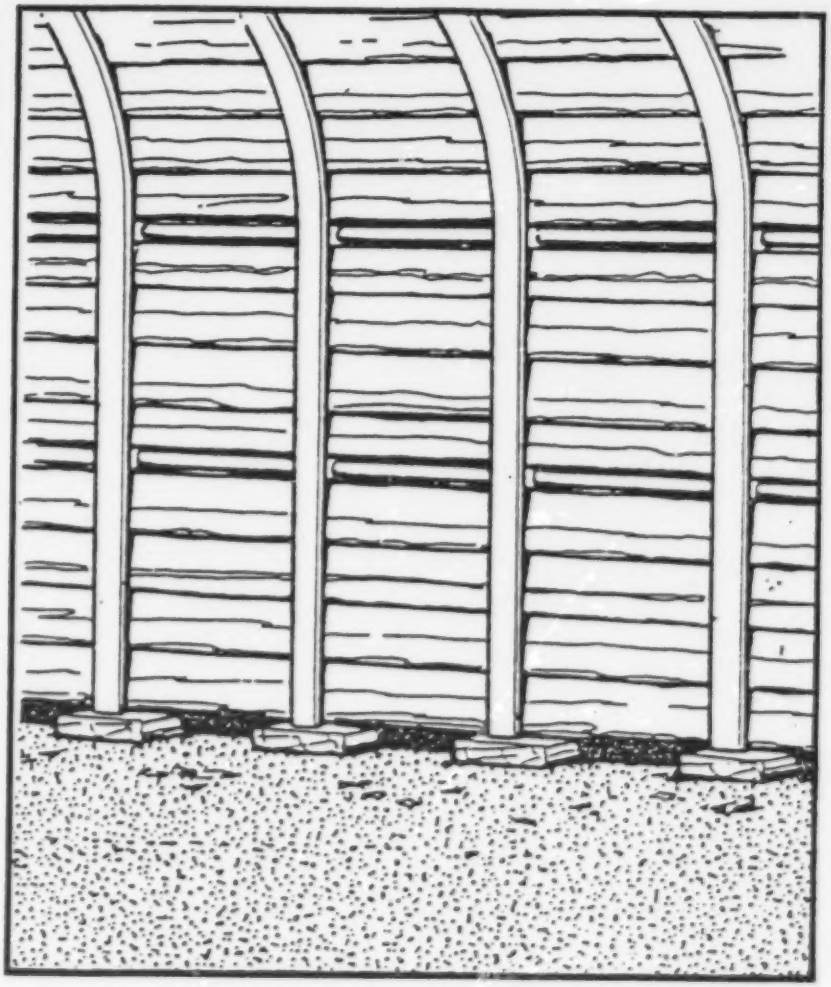
Date MAY 26, 1988
Project 871-1369

Golder Associates

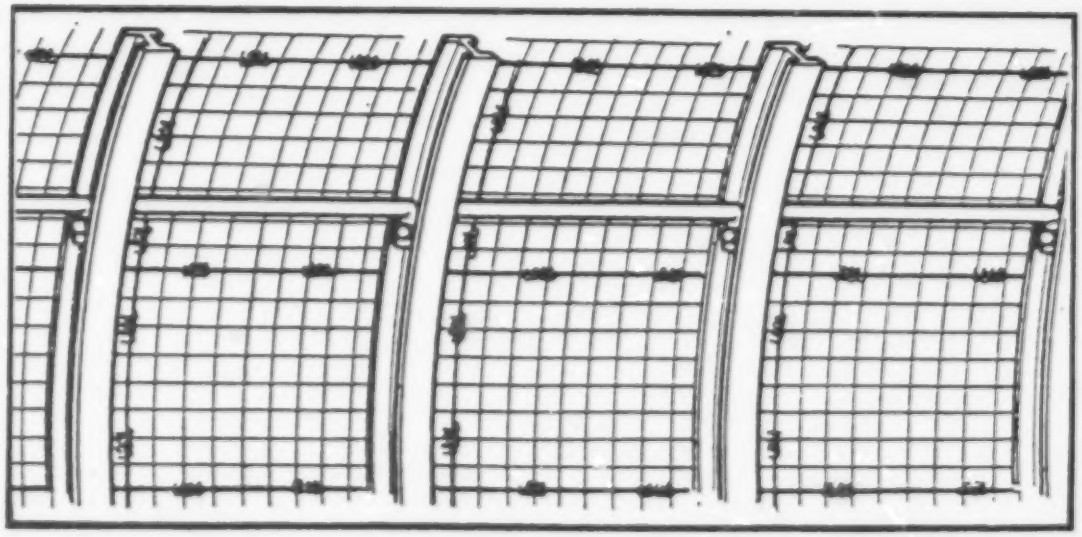
Drawn D. M.
Chkd F. A.

EXAMPLE OF LAGGING

FIG. 2.10



TIMBER BOARDS



WIRE MESH

Date MAY 26, 1988

Project 871-1389

Golder Associates

Drawn D.M.

Chkd F.M.

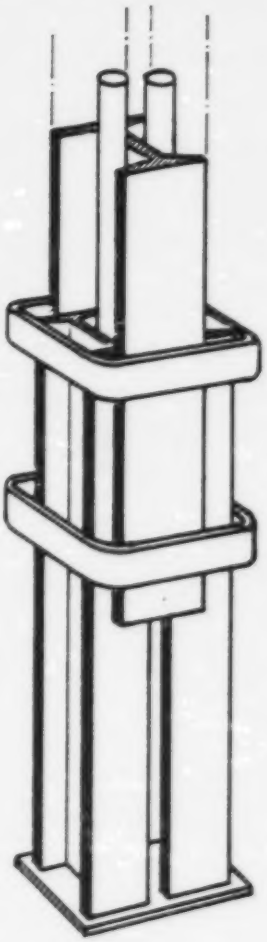
DRG No A 871 1037 010

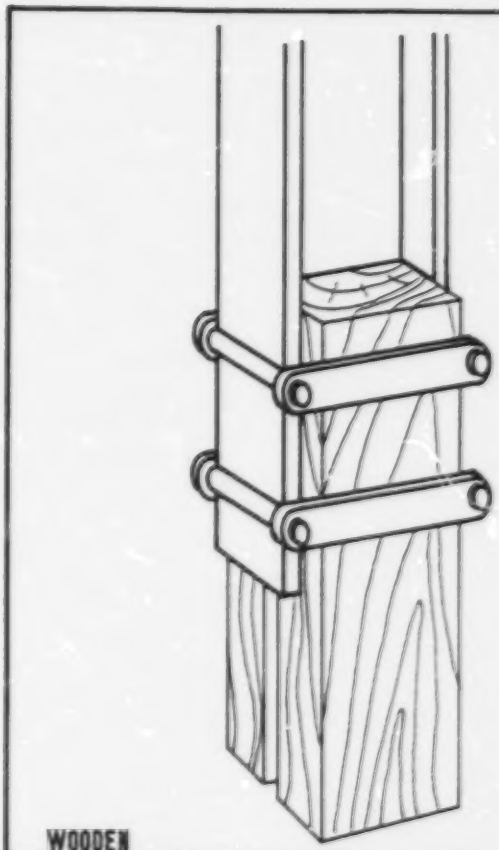
REVIEWED BY

DATE MAY

EXAMPLE 'OF STILTS

FIG. 2.11





Date MAY 26, 1988

Project 871-1369

Golder Associates

Drawn D.M.

Chkd

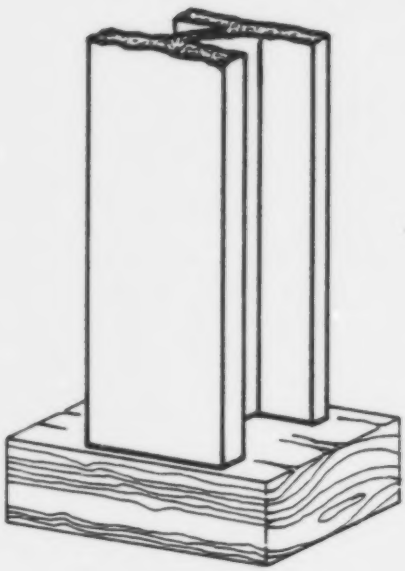
DRG. NO A 103 1

REVIEWED BY

DATE MAY

FOOTPIECE

FIG. 2.12



Date APRIL 25, 1988

Project 871-1369

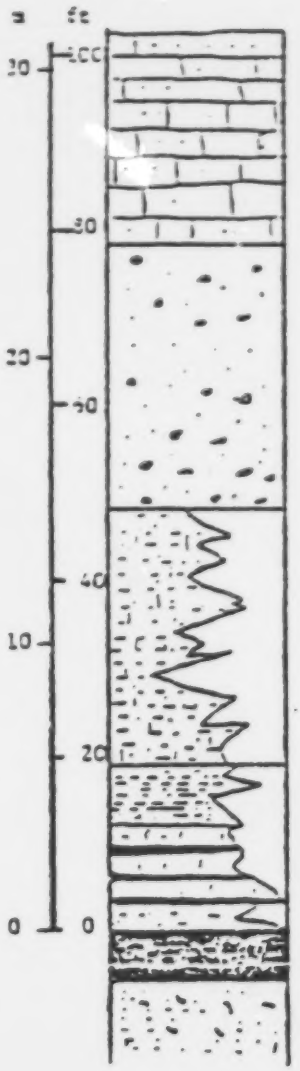
Golder Associates

Drawn D. M.

Chkd. F. M.

GENERAL COLUMNAR SECTION OF MINE ROOF-ROCK
(AFTER UNAL 1983)

FIG. 2.13



Limestone 30 ft (9.2 m)

Claystone with limestone nodules calcareous mudshale or sandy shale. 19-30 ft (5.8-9.2 m)

Sandy shale, shale with interbedded sandstone or sandstone, 13-50 ft (3.5-15.2 m)

Black shale, 0-6 ft (0-2.4 m)

Interbedded mudshale and coal 0-8 ft (0-2.4 m)

Drawslate

Pittsburgh coalbed, 6 ft (1.3 m)

Calcareous claystone

Date APRIL 25, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd. F.M.

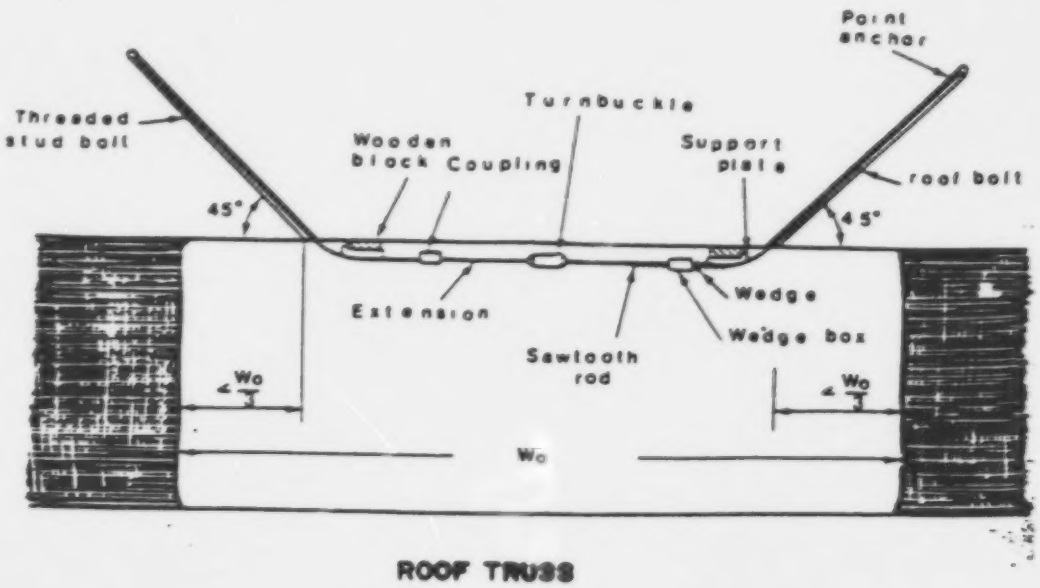
DRG. No. A 875 1037-012

REVIEWED BY

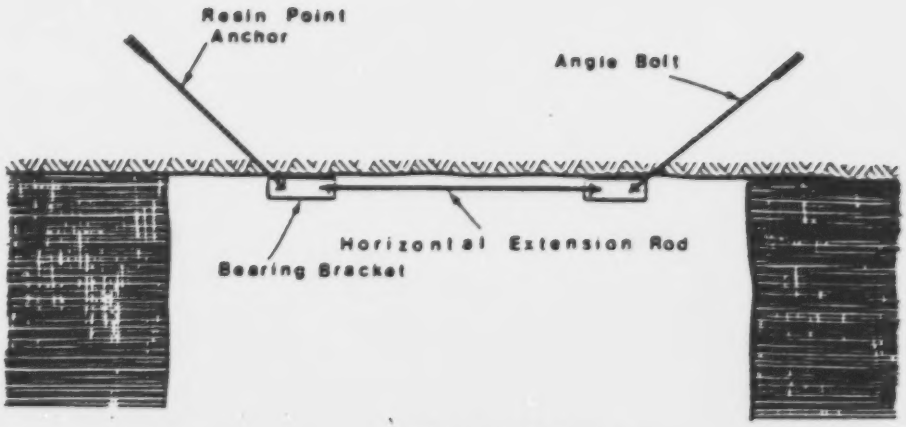
DATE MAY 88

ACTIVE SUPPORT-TYPICAL ROOF TRUSS (AFTER PENG 1986)

FIG. 2.14



ROOF TRUSS



The angle-bolt truss.

Date MAY 26, 1988
Project 871-1369

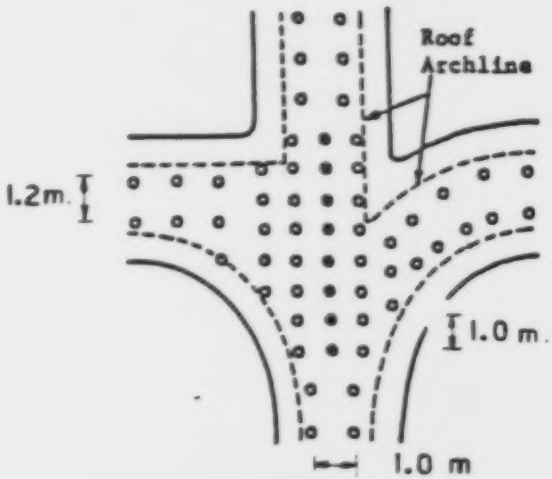
Golder Associates

Drawn D.M.
Chkd F.M.

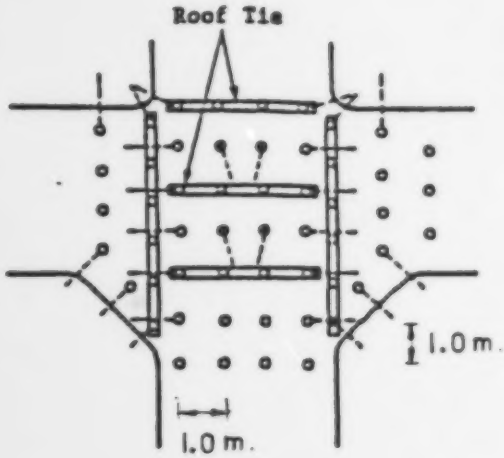
DAG NO. 8[illegible]037
REVIEWED BY
DATE MAY

ROCK BOLTING PATTERN
(AFTER STAHL 1962)

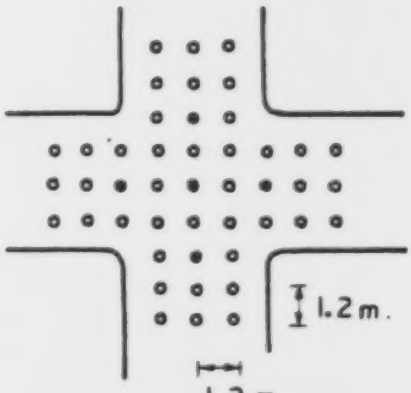
FIG. 2.15



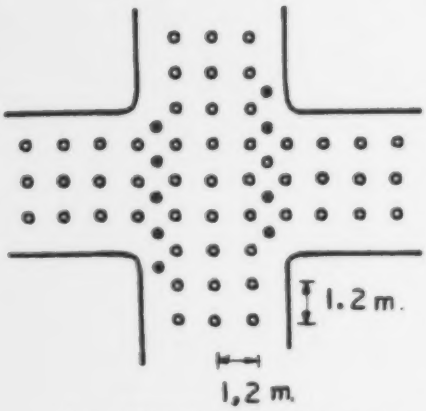
a. Boring-Type Miner



b. Ripper-Type Miner



c. Ripper- and Boring-Type Miner



d. Ripper-Type Miner or Conventional Loading

o Regular Bolts
• Longer Bolts 0.3m. LONGER THAN REGULAR BOLTS
o--- Angled Bolts

Date APRIL 21, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.M.

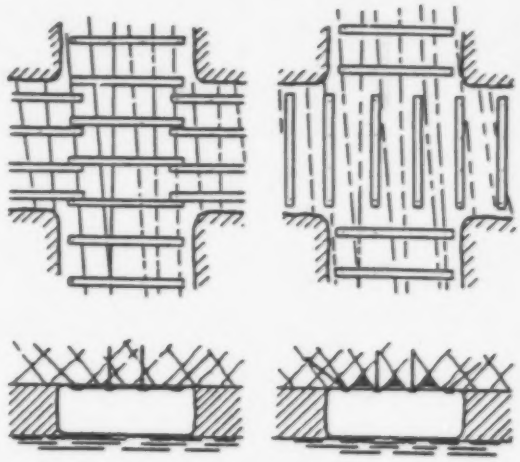
DRG. No. A 875 1037-012

REVIEWED BY

DATE MAY 30

ROCK BOLTING SUPPORT
(AFTER GERDEEN ET AL 1977)

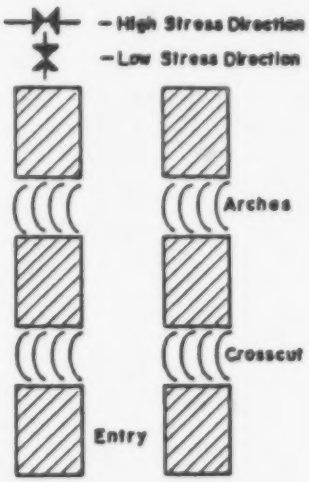
FIG. 2.1



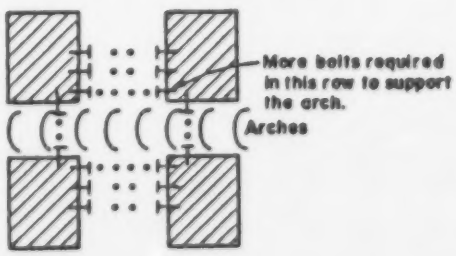
a. Good coverage of slips everywhere.

b. Poor coverage of slips in the intersection.

a. PLANKS COMBINED WITH ROOF BOLTS



a. Discontinuous Ground Arch at Intersections



b. Additional Support Along the Edges of an Intersection

Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd FA.

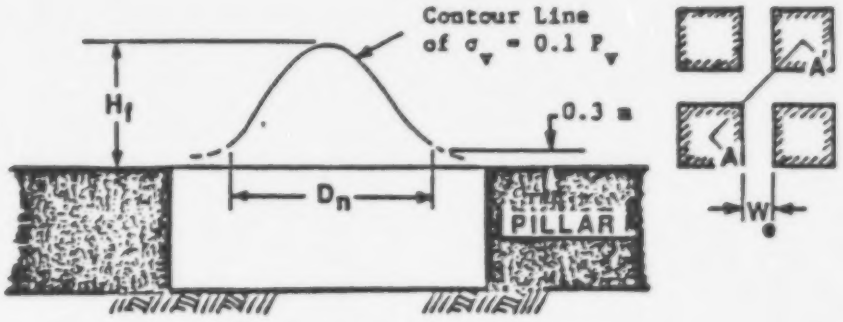
DRG No A 875 1037-012

REVIEWED BY

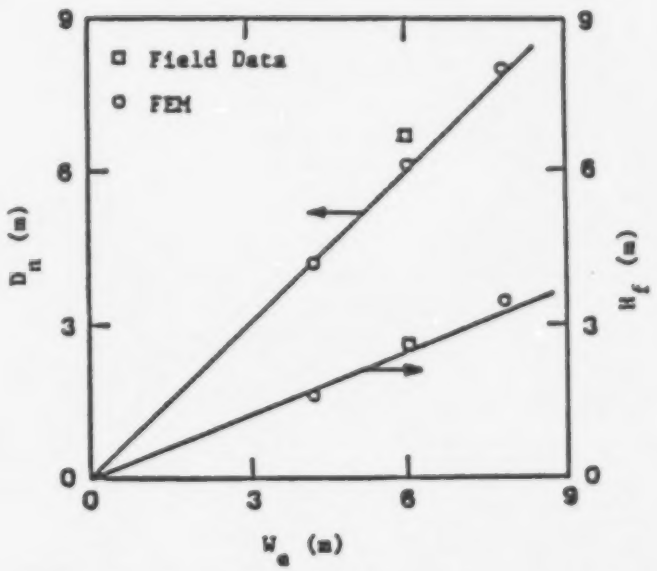
DATE MAY

LOOSENED ZONE
(AFTER PENG ET AL 1978)

FIG. 2.17



Section A-A'



D_n = Nominal Bottom Dimension of Roof Fall, $D_n = (D_{max} \times D_{min})^{1/2}$

H_f = Average Height of Roof Fall, $H_f = 0.37\ D_n$

W_e = Width of Entry

P_v = Vertical in situ stress

σ_v = Vertical Induced Stress

Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.M.

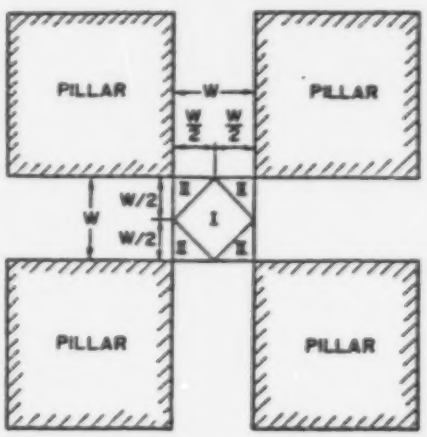
DRG. No. A 875 1037-012

REVIEWED BY

DATE MAY 26

ROCK BOLTING PATTERN
SUSPENSION ACTION
(PENG ET AL 1978)

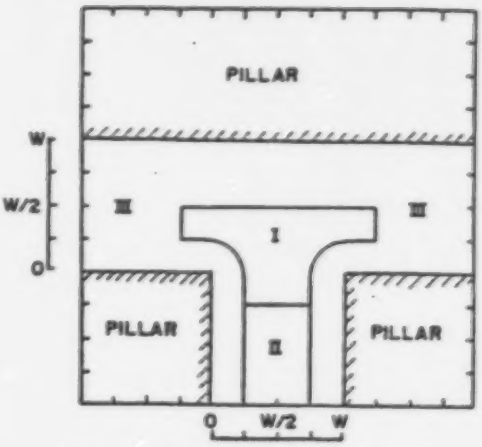
FIG. 2.18



Region	Bolt Length	Carrying Capacity
I	0.5 W	0.5 γ_r W
II	0.25 W	0.25 γ_r W

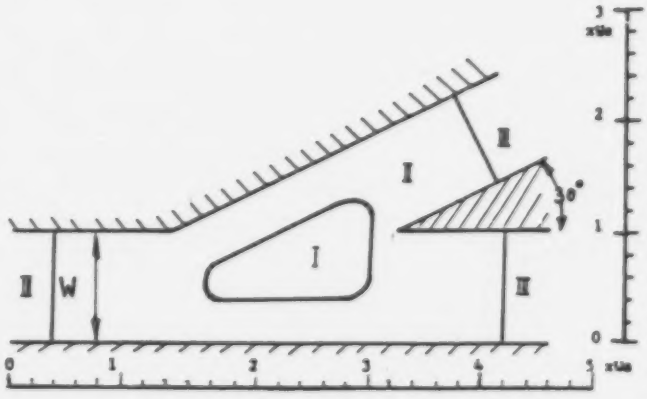
(γ_r = Unit Weight of Roof Rock)

A) FOUR-WAY (+) INTERSECTION



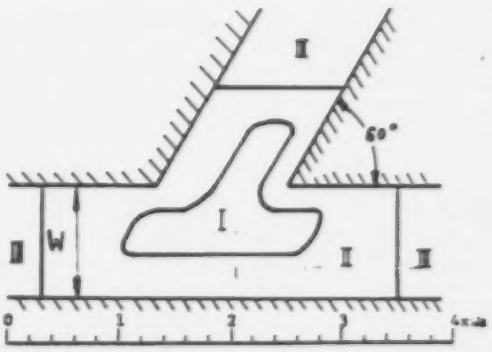
Region	Bolt Length	Carrying Capacity
I	0.4 W	0.4 γ_r W
II	0.3 W	0.3 γ_r W
III	0.2 W	0.2 γ_r W

B) THREE-WAY (T) INTERSECTION



REGION	BOLT LENGTH	CARRING CAPACITY
I	0.5 W	0.5 γ_R W
II	0.3 W	0.2 γ_R W
III	0.15 W	0.5 γ_R W

C) THREE-WAY (30°) INTERSECTION



REGION	BOLT LENGTH	CARRING CAPACITY
I	0.4 W	0.4 γ_R W
II	0.2 W	0.2 γ_R W
III	0.15 W	0.15 γ_R W

D) THREE-WAY (60°) INTERSECTION

Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.A.

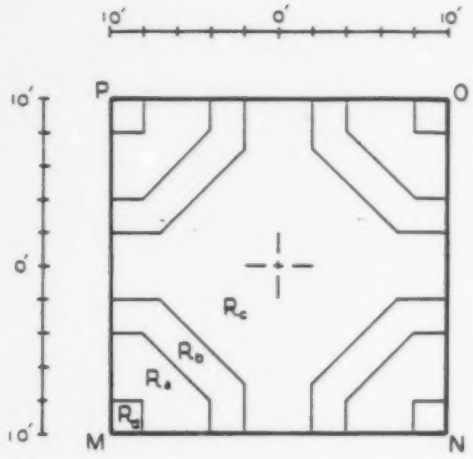
DRG. No A 875 1037-012

REVIEWED BY

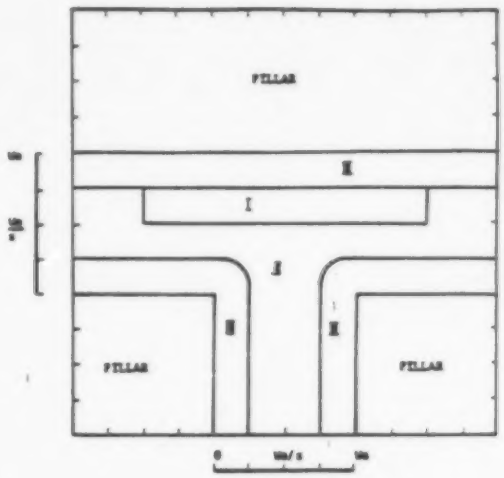
DATE MAY 88

ROCK BOLTING PATTERN SUGGESTED REINFORCEMENT ACTION (PENG ET AL 1978)

FIG. 2.19



	S_r kip/ft²	S_r / σ_o
R_a	4 – 5	0.045 – 0.056
R_b	2 – 4	0.022 – 0.045
R_c	0 – 2	0 – 0.022
R_d	STABLE	STABLE

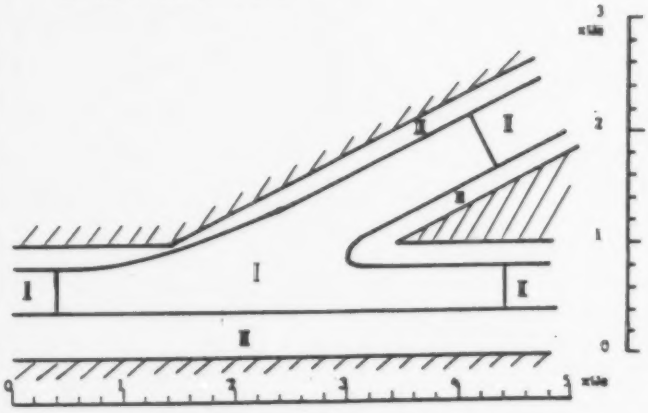


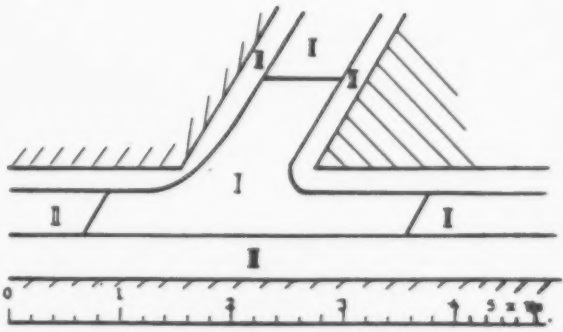
REGION	BOLT LENGTH	REQUIRED SHEAR RESISTANCE	REQUIRED PRETENSION
I	l_b	S_r	P
II	0.8 l_b	0.5 S_r	0.5 P
III	0.6 l_b	0.3 S_r	0.3 P

A) FOUR – WAY (+) INTERSECTION

B) THREE – WAY (+) INTERSECTION

S_r = SHEAR RESISTANCE (Kip/ft²)
σ_o = OVERBURDEN STRESS (lbs/ft²)
P = PRETENSION (lbs/ft²)
l_b = BOLT LENGTH (ft)





REGION	BOLT LENGTH	REQUIRED SHEAR RESISTANCE	REQUIRED PRETENSION
I	l_b	S_r	P
I	0.8 l_b	0.4 S_r	0.4 P
II	0.6 l_b	0.2 S_r	0.2 P

REGION	BOLT LENGTH	REQUIRED SHEAR RESISTANCE	REQUIRED PRETENSION
I	l_b	S_r	P
II	0.8 l_b	0.5 S_r	0.5 P
III	0.6 l_b	0.3 S_r	0.3 P

C) THREE – WAY (30°) INTERSECTION

D) THREE – WAY (60°) INTERSECTION

Date MAY 26, 1988
Project 871 - 1369

Golder Associates

Drawn D.M.
Chkd F.M.

DRG No. A 875 1037 – 012

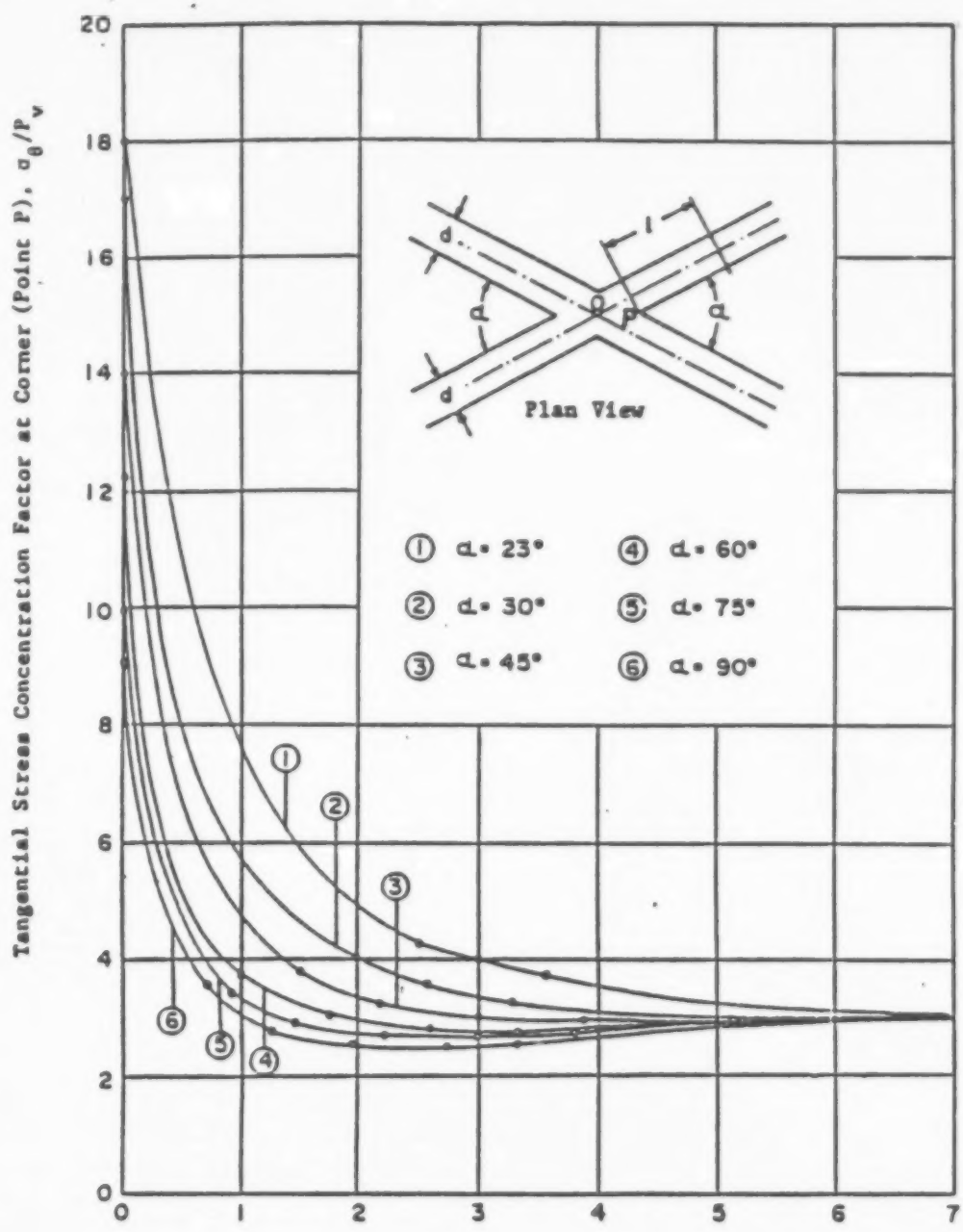
REVIEWED BY

DATE MAY

PLOT OF THE RESULTS OBTAINED WITH PHOTOELASTICITY ANALYSIS

(AFTER KANESHIGE ET AL 1964)

FIG. 2.20



Variation of the Stress Concentration Factors with the Normalized Distance from the Corner of Model Intersection in Uniaxial Stress Field

Date APRIL 25,1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd F.M.

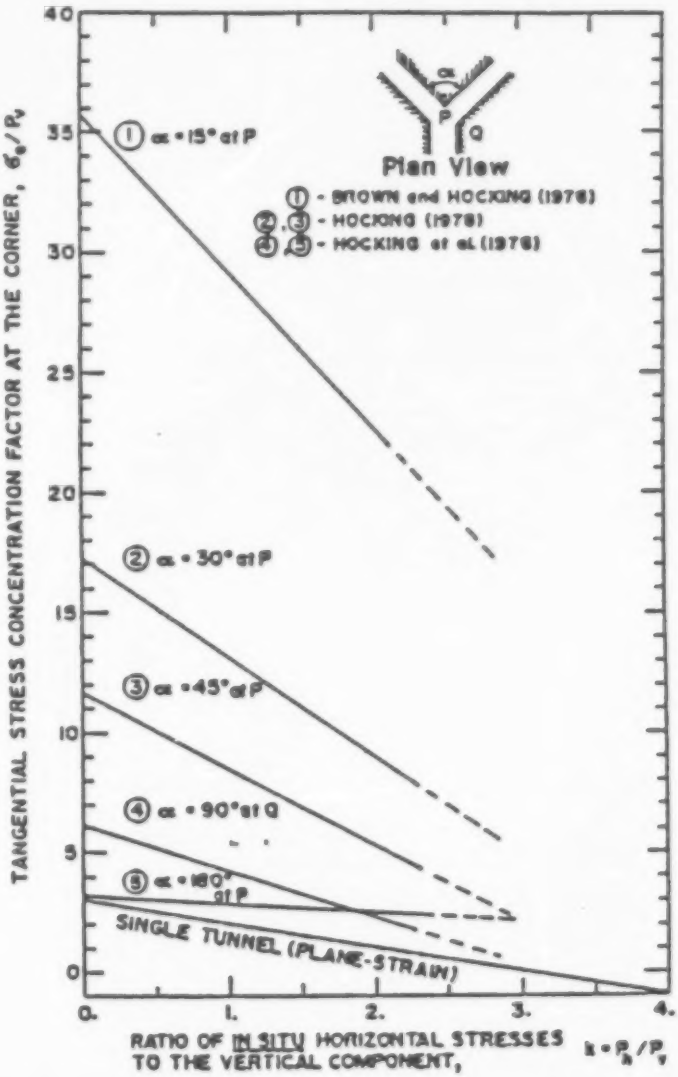
DRG No A 875 1037-012

REVIEWED BY

DATE

PLOT OF THE RESULTS FROM BOUNDARY-ELEMENT ANALYSIS
(AFTER HOCKING ET AL 1976)

FIG. 2.21



Variation of the Tangential Stress Concentration Factor at the Mid-Height of Bifurcation Corner for Various Field Stresses (Data after Hocking et al., 1976; Brown and Hocking, 1976; Hocking, 1978).

Date APRIL 25, 1988
Project 871-1369

Golder Associates

Drawn D M
Chkd F.M.

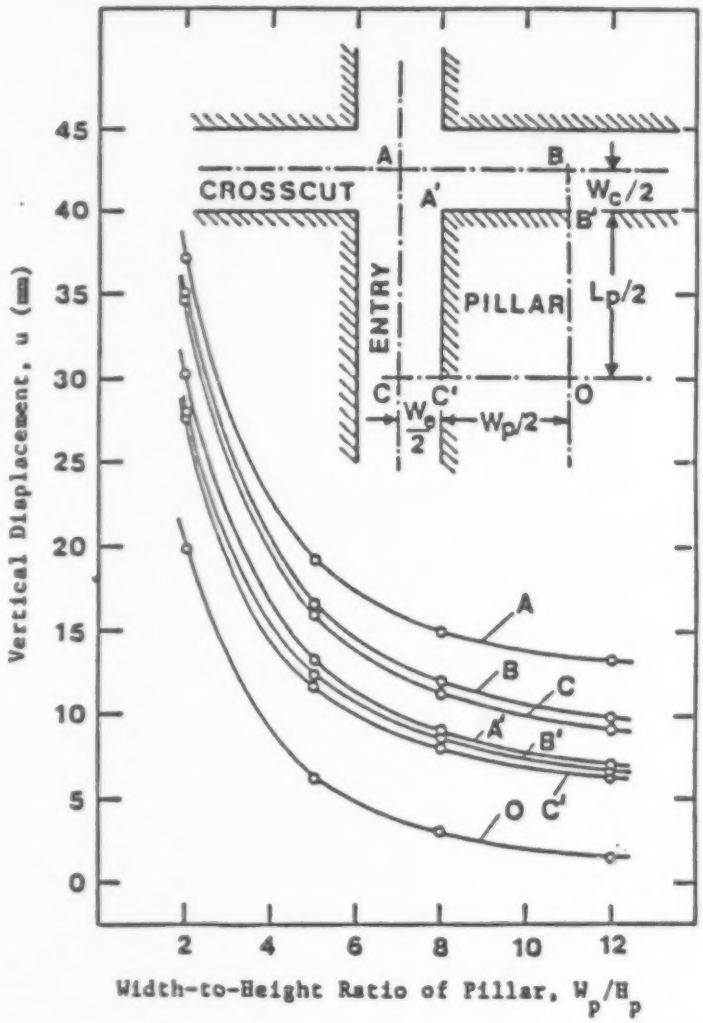
DRG No A 875 1037-012

REVIEWED BY

DATE MAY 89

PLOT OF VERTICAL DISPLACEMENT
(AFTER GERCEK 1982)

FIG. 2.22



Vertical Displacement of the Roof Plane at Some Selected Points ($k = P_h/P_v = 0.5$)

WHERE: W_c = CROSSCUT WIDTH

W_e = DRIFT WIDTH

W_p = PILLAR WIDTH

L_p = PILLAR LENGTH

Date APRIL 25, 1988
Project 871-1369

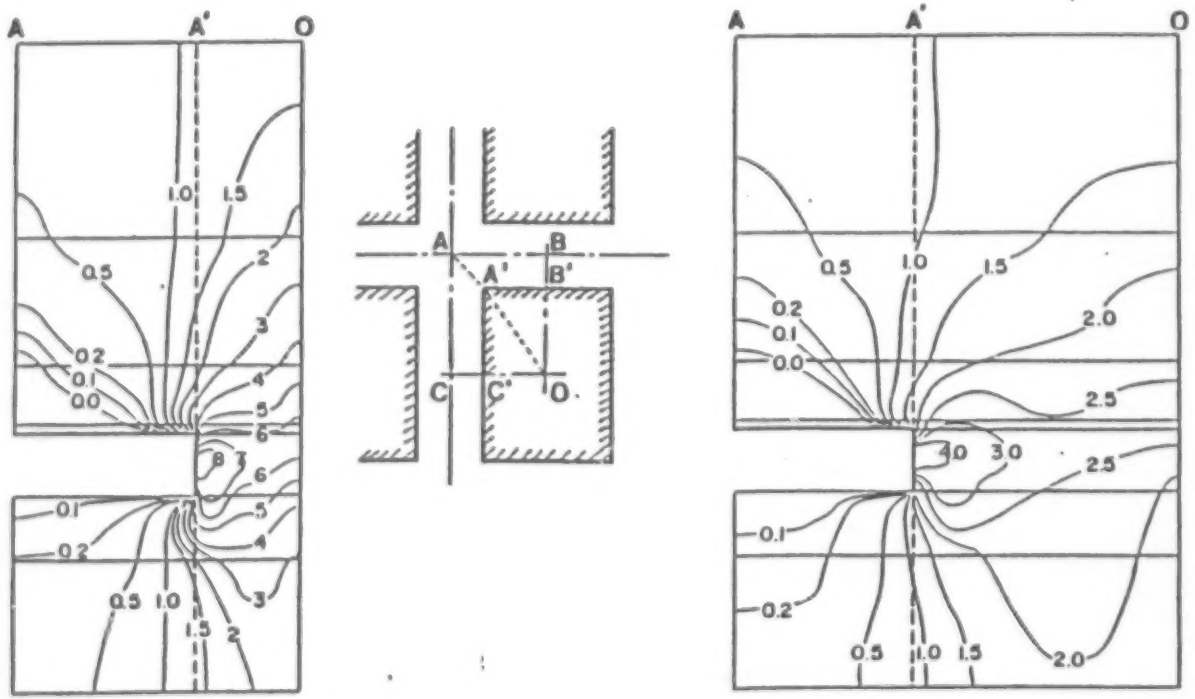
Golder Associates

Drawn D. M.
Chkd

DRG No A 875 1037-012

Golder Associates

VERTICAL STRESS DISTRIBUTION
(AFTER GERCEK 1982)



a. Model 1 (W_p/H_p = 2.0)
$k = P_h/P_v = 0.5$, $(\sigma_p/P_v)^* = 7.50$

b. Model 2 (W_p/H_p = 5.0)
$k = P_h/P_v = 0.5$, $(\sigma_p/P_v)^* = 2.88$

$(\sigma_p/P_v)^*$: Average Normalized Pillar Stress According to the Tirbutary-Area Theory.

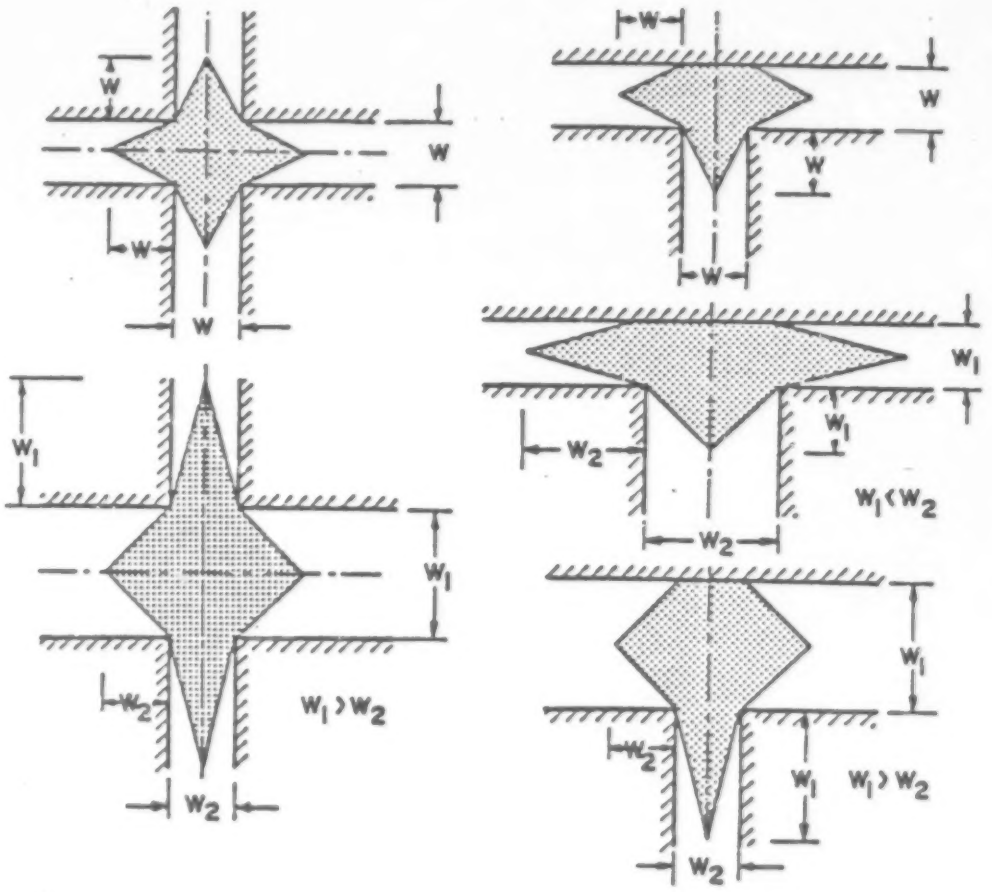
Distribution of the Vertical Stress Concentration Factors in the Diagonal Planes Passing Through the Center of Intersection, Pillar Corner, and Pillar Center

FIG. 2.23

Drawn D.M.
Chkd F.M.

AREA OF SPECIAL ROOF BOLTING PATTERN (AFTER GERCEK 1982)

FIG. 2.24



a. "+"- Intersections

b. "T"- Intersections

The Support-Revision Area for Coal-Mine Intersections.

Date MAY 26, 1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd R.M.

DATE MAY 30
REVIEWED BY
DRG No A 875 1037-012

ROOF BOLTING PATTERN GUIDELINES
(AFTER UNAL 1983)

FIG. 2.25

Roof-support design chart for coal mining – Intersection of two entries, Entry width: 20-ft

ROCK CLASS	ROCK MASS RATING RMR	ROCK LOAD HEIGHT H_R (FT)	SUPPORT SPECIFICATIONS		ALTERNATE SUPPORT - PATTERNS	
			MECHANICAL BOLTS/POSTS	RESIN BOLTS/POSTS	MECHANICAL BOLTS/POSTS	RESIN BOLTS/POSTS
I VERY GOOD	90	2.8	L : 2.5' S : 5' x 5' G : 40 ø : 3/4" C : 8.8 tons	L*: 3.073.0' S : 5' x 5' G : 40 ø : 3/4" C : 8.8 tons		
	80	5.7	L : 3.0' S : 5' x 5' G : 40 ø : 5/8" C : 8.2 tons $ø_p$: 4"	L*: 4.0'/4.0' S : 5' x 5' G : 60 ø : 1" C : 23.7 tons $ø_p$: 4"		
II GOOD	70	8.5	L : 5.0' S : 5' x 5' G : 40 ø : 3/4" C : 8.8 tons $ø_p$: 5"	L*: 3.075.0' S : 5' x 5' G : 60 ø : 3/4" C : 13.2 tons $ø_p$: 4.3"		
	60	11.3	L : 4.0' S : 5' x 5' G : 40 ø : 3/4" C : 8.8 tons $ø_p$: 4"	L*: 3.576.0' S : 5' x 5' G : 60 ø : 3/4" C : 13.2 tons $ø_p$: 5.5"		
III FAIR	50	14.2	L : 7.0' S : 5' x 5' G : 40 ø : 5/8" C : 8.2 tons $ø_p$: 7"	L*: 3.576.0' S : 5' x 5' G : 60 ø : 3/4" C : 13.2 tons $ø_p$: 6.5"		
	40	17.0	L : 9.0' S : 5' x 5' G : 40 ø : 5/8" C : 8.2 tons $ø_p$: 5.5"	L*: 3.577.0' S : 5' x 5' G : 60 ø : 1" C : 15.8 tons $ø_p$: 7"		
IV POOR	30	19.8	L : 10.0' S : 5' x 5' G : 40 ø : 5/8" C : 8.0 tons $ø_p$: 6.0"	L*: 4.578.0' S : 5' x 5' G : 60 ø : 1" C : 23.7 tons $ø_p$: 7"		
	20	22.6	L : 12.0' S : 5' x 6' G : 40 ø : 5/8" C : 5.0 tons $ø_p$: 6.5"	L*: 5.078.0' S : 5' x 6' G : 60 ø : 1" C : 23.7 tons $ø_p$: 7"		

WHERE : L=BOLT, S=BOLT SPACING, G=GRADE OF STEEL, Ø=BOLT DIAMETER
C=BOLT CAPACITY, ØP=POST DIAMETER, L*=RESIN LENGTH/BOLT LENGTH

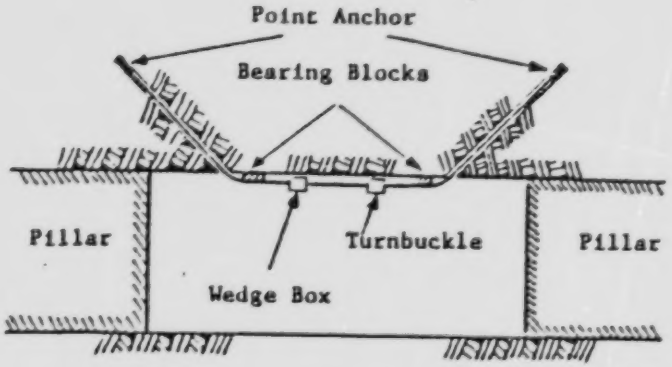
Date APRIL 21, 1988
Project 871-1369

Golder Associates

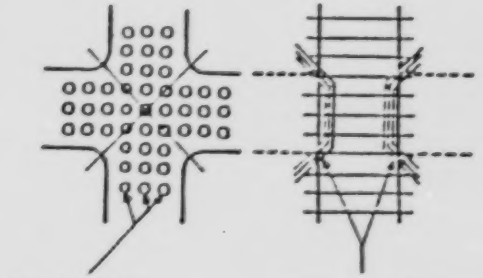
Drawn D.M.
Chkd P.M.

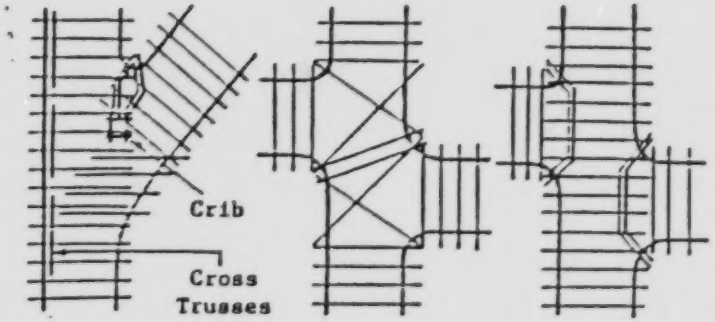
DATE MAY 88
REVIEWED BY
875 1037-012

SUGGESTED INTERSECTION SUPPORT (AFTER GERCEK 1982)

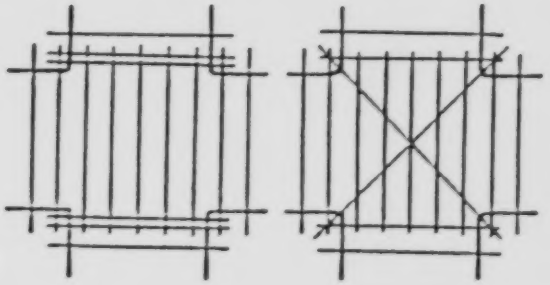


a. Roof-Truss Assembly





b. Three-Way Intersections



c. Four-Way Intersections

Special Intersection-Support Suggestions Using Roof-Truss Assemblies

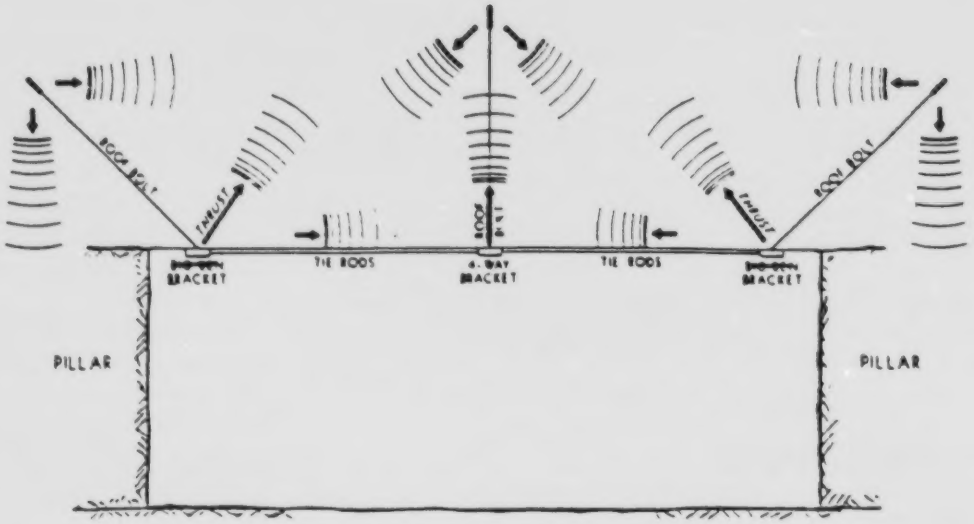
FIG. 2.26

Golder Associates

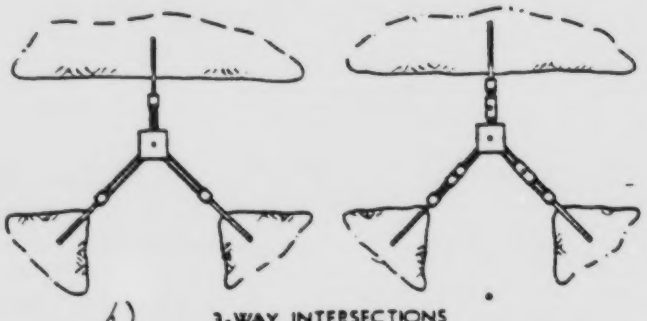
Date MAY 26, 1988
Project 871-1369
Drawn D.M.
Chkd [illegible]

NEW TRUSS BOLTING SYSTEM
(AFTER HARAMY 1988)

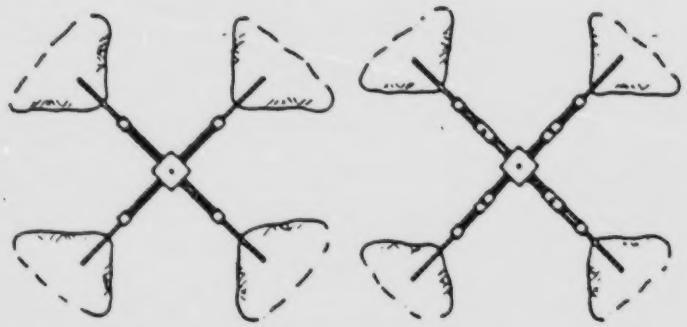
FIG. 2.27



INTERSECTION TRUSS ASSEMBLY AND FORCE DISTRIBUTION



b) 3-WAY INTERSECTIONS



c) 4-WAY INTERSECTIONS

NEW INTERSECTION-TRUSS BOLT SYSTEM

Date MAY 26, 1988
Project 871-1369

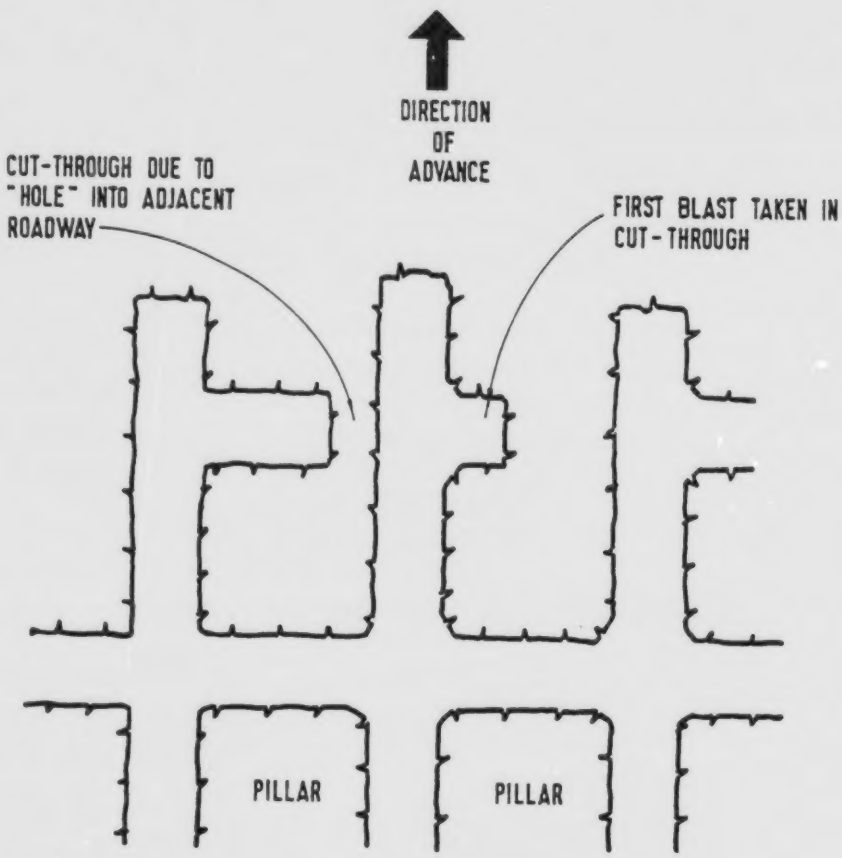
Golder Associates

Drawn D.M.
Chkd [illegible]

DATE MAY 89
REVIEWED BY
875 1037-012

METHOD OF FORMING JUNCTION IN SOUTH AFRICA BORD AND PILLAR MINING

FIG. 2.28



Date APRIL 25, 1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd F. M

No. A 875 1037 - 012 REVIEWED BY JWS DATE MAY 86

Date APRIL 25, 1988

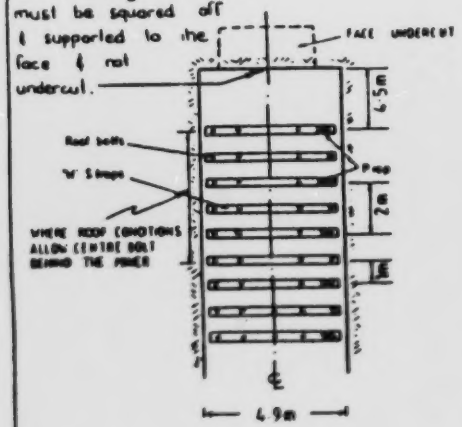
Golder Associates

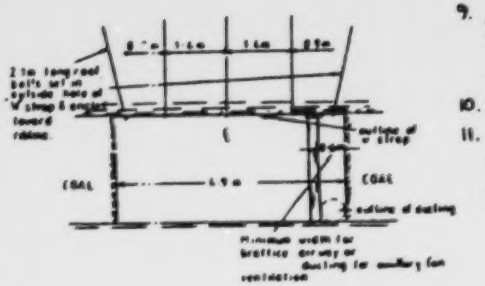
Drawn D.M. Chkd F.M.

SUPPORT RULE NOTES FOR ALL DEVELOPMENT DRIVAGES

HEADINGS & CUT-THROUGHS

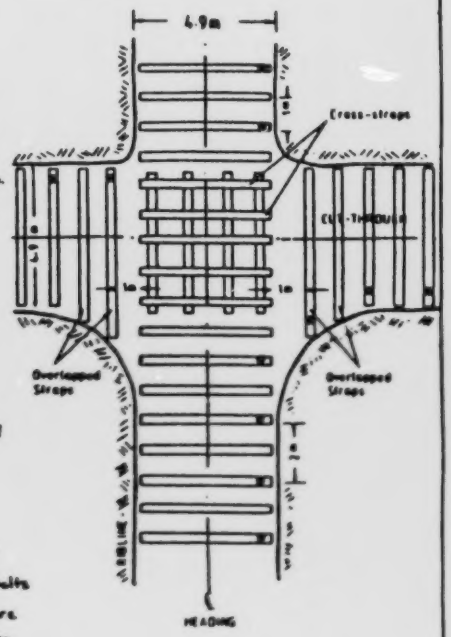
At the completion of each drivage the place must be squared off & supported to the face & not undercut.




2.1m long roof bolts set on radius here at 'W' strap & angled toward ribline.
outline of 'W' strap
COAL
4.9m
COAL
outline of sealing
Minimum width for trafficway or way of ducting for auxiliary fan ventilation

1. This plan indicates MINIMUM TIMBERING and Support. Nothing in these support rules shall prevent a workman from setting supports in his working place at more frequent intervals than those specified on this plan.
2. Roof support system to be STRICTLY ADHERED to.
3. MINER CANOPY not to proceed beyond last bolted support.
4. PROPS to be placed on one side of the heading for VENTILATION purposes. (TEMPORARY).
5. All DRIVAGE WIDTHS are to be 4.9m (Max. width is 6.5m). All other distances shown are maximum
6. All drivages are to be kept ON CENTRE.
7. RIB SUPPORT :- (i) to be placed where necessary or under instruction from an undermanager. (ii) to consist of strip mesh and 0.9m bolts.
8. FOR ALL DEVELOPMENT DRIVAGES 4.9m long straps are to be set at 1.0m centres, together with FIVE 2.1m long full column chemical anchor roof bolts
9. INTERSECTION SITE to be cross-stopped before intersection is formed. Minimum of three straps with five bolts per strap to be used for cross-stopping
10. STUBS longer than 10m must be ventilated.
11. When R.S.J.'s are to be used:-
    (i) a softwood cap shall be set over each prop.
    (ii) a roof bolt through each end shall be used to secure them.

INTERSECTIONS



COLLIERY MANAGER Brian Nicholls Date 17/10/'84

TAHMOOR MINE			
Minimum Timbering Diagram For Pillar Development.			
Drawn R.H.M.		A3	Drawing No. TC/34
Checked	Issued		
Date 1-2-84	Issue HTS		

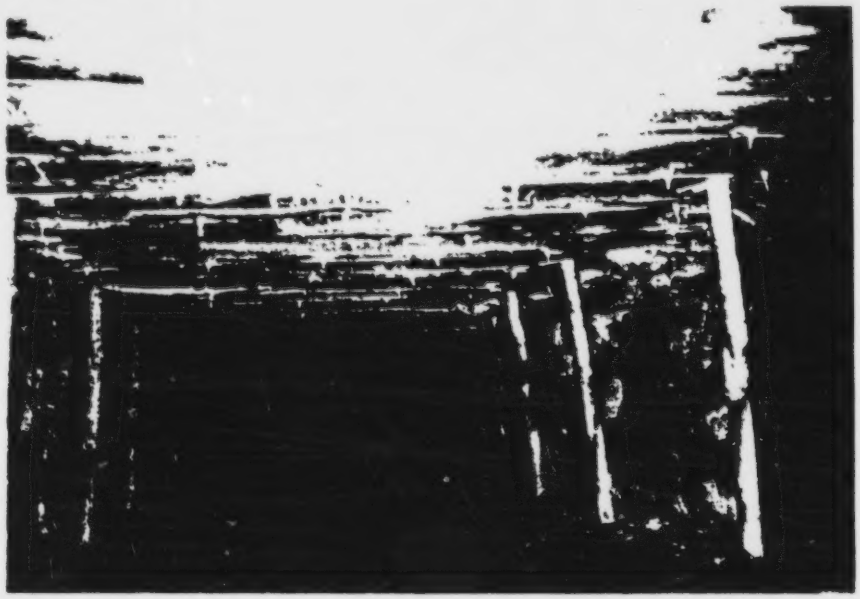
THIRD GENERATION SUPPORT, AUSTRALIA (AFTER AEBBLEWHITE 1986)

FIG. 2.29

Support rule notes for all development drivages, Tahmoor Colliery, 1985.

TYPICAL SUPPORT PATTERN
THIRD GENERATION SUPPORT

FIG. 2.30



Third Generation Support.
Typical roof support for development roadways.

REF: AFTER AEBBLEWHITE 1986

DATE MAY 32

REVIEWED BY

DWG No A 875 1037-012

Date APRIL 25, 1988
Project 871-1369

Golder Associates

Drawn D M
Chkd

3. EXISTING CONDITIONS IN THE SYDNEY COALFIELD

Currently, the three producing mines in the Sydney coalfield are the Lingan, Prince and Phalen Mines. Development in these mines have extended to about 600, 250 and 450 m below sea level respectively. On the basis of the data available from CBDC and our discussion with the Civil Engineering and Geological Department of CBDC, the rock conditions, the support system and the reported/ observed performance of the installed supports are summarized in the following sections. For the purpose of this study, the Lingan Mine, being the deepest and the largest producing mine in the Sydney Coalfield, was selected as the reference mine for detailed review and evaluation with regard to roadway and roadway intersection supports. Hence, more information relating to the roadway intersection supports in the Lingan Mine were included.

3.1 Lingan Mine

The Lingan Mine is located near New Waterford. The mining development area extends approximately 3.5 km offshore in a near northerly direction. The workable coal seam at the mine is the Harbour seam which is approximately 1.8 m thick and dips at about 13 percent to the north.

In general, the coal seam is overlain by a thin mudstone seam followed by a sequence of siltstone, sandstone and limestone above. Below the coal seam, the strata consists mainly of interbedded siltstone and mudstone with occasional thin layers of sandstone. The average mechanical properties of the rock are summarized in Table 3.1. Overcoring and hydro-fracturing stress measurements have been undertaken in Lingan Mine (Golder Associate Report No. 861-6015) and the data are summarized in Table 3.2.

The main roadway (slope) dimensions are 6 m wide x 3.7 m high and they are developed to a horseshoe shape with the base of the coal seam at the floor level. The roof of the roadway is in either mudstone or siltstone depending on the local variations in the thickness of the mudstone and siltstone layers. Based on available geological data, the mudstone could vary from 0.2 to as much as 1.5 m.

The roadway intersections in Lingan mine are generally situated in the principal system of haulage and airways in the vicinity of the entrie to the production longwall panels. The main roadways are generally separated by barrier pillars of about 65 m. Most often, the four-way intersections are formed by the junction of one of the main slopes (main roadway running in a northerly orientation) with a crosscut, a tail gate or a headgate entry. The three-way intersections which are generally of two shapes, T and Y, are located closer to the working face. The Y junction is generally used to create a by-pass and the T junctions are often used at the intersection of the main slopes and a tailgate or headgate entry and between the entries and the pillar barrier way.

3.1.1 Support for Main Roadways and Intersections

The main roadways at Lingan Mine were excavated in the past by drill-and-blast method with the coal taken out first followed by excavation of the strata above. In recent years, the excavation method was improved with the utilization of road header machines. The roadways are generally supported by steel sets consisting of 114 x 127 mm x 29 kg/m arched sections spaced at 1 m centres. These sets are generally erected at about 3 m behind the face during development of the roadway.

Timber lagging is used to block the steel set generally at the roof arch (approximately 20° to 50° from arch apex). Occasionally, the lagging was brought down to the sides at intersections and in areas where substantial overbreak and loosening had occurred. Steel corrugated sheets of 1.2 m x 2 m are installed between the steel sets over the arched portion.

At the intersections, a variety of support systems are adopted. For three and four-way intersections, the dimensions of the cross cuts are generally about the same as those of the main slopes. Square work is typically used at such intersections, a series of roof beams are generally used to transfer the rock load to collar beams which span across the slope. Figures 3.1 to 3.3 show the general arrangement of the structural members of three and four way intersections. The sizes and properties of the structural members are summarized in Table 3.3. For all intersections, the cross cuts were excavated after the supports in the main slopes had been installed.

A series of photographs (plates 1 to 9) were taken at typical four-way and three-way intersections to illustrate the configuration of the support system installed. The locations where the photographs were taken are shown in Figure 3.4.

3.1.2 Support Performance

In connection with this study, a visit was made to the Lingan Mine to observe the current conditions of the installed support system in the main roadways (slopes) and in particular, at the roadway intersections. The visit was conducted in November 1987 and a separate report on the visit had been submitted to the Cape Breton Coal Research Laboratory. A copy of the site visit report is attached in Appendix A.

Based on the field observations and subsequent discussions with the mining and geological personnel of CBDC, the current condition and performance of the support systems in the main roadways and roadway intersections are summarized below.

- There are very few discontinuities other than lithological bedding surfaces encountered in the slopes and intersections.

- The stratigraphic structure above the coal seam generally comprises the following: a mudstone layer ranging from 0.2 m to 1.5 m thick overlain by a thick bed of siltstone which consists of two distinct zones, a massive and a layered (laminated) zone. The sequence of these two zones can be reversed along the coal seam. A thick bed of massive sandstone generally overlies the siltstone layers.

- The RQD (Rock Quality Designation) values of the rock are as follows:

Rock Type	RQD
- layered (laminated) siltstone	80%
- massive siltstone	100%
- massive sandstone	100%
- average condition of rock over the coal seam	80%

- Approximately 70 percent of the roof of the roadway intersections were excavated in the siltstones. It was reported that rock loosening was generally more extensive where mudstone and layered siltstone were encountered at the roof. Massive sandstone and siltstone roofs were generally stable. Fracturing in these materials was possibly accentuated by blasting.

- Where rock loosening was observed in the roadways, the zones of loosening were characterized by dome shapes varying from 0.2 m to as much as 2.5 m in the worst case. The average height of loosening is about 1 m. Progressive loosening was likely attributed to the separation of bedding surfaces as the installed support system did not provide active support to the rock.

- The formation of the intersections tends to produce brows of poorly supported material due to the lack of lagging or excessive deflection of the collar beams

- In general, no formal measurements have been made of the deflections of roadway arches and structural beams at the intersections.

- At three-way and four-way intersections, where flat work supports are used, only minor deflections were observed in the roof beams. However deflection of the order of 50 to 70 mm were observed in most collar beams which support the roof beams. Deflection in excess of 150 mm was noted at one location (Plate 5). This was attributed to the loosening of the bolted joint between the collar beam and the supporting column. At this location, some settlement of the column was also observed.

- No noticeable deflections were observed in the arches in the main roadways.

- At some three-way intersections where arch supports were continued from the roadways into the intersection (see plate 8 and 9), the collar beam was supported by pairs of rock bolts at 0.8 m spacing. These bolts are 19 mm dia. and 1.5 m long

connected by a 300 mm x 150 mm x 10 mm plate. In general, very minor deflections were observed in these beams.

3.2 Prince Mine

Prince Mine is located at Point Aconi near North Sydney. The mining development area extends approximately 3 km offshore in a northerly direction. The workable coal seam at the mine is the Hub seam which is approximately 2.2 m thick and dips gently to the north at 3 to 5 degrees. The coal seam is currently mined by retreat longwall mining techniques.

In general, the coal seam is overlain by mudstone with thicknesses ranging from 25 mm to 4 m. Above the mudstone, there is a sequence of interbedded siltstone and sandstone, and shale. Paleo-river sandstone channels are often present above the mudstone layers and have created quite variable roof conditions.

The contact between the mudstone and interbedded siltstone and sandstone is generally weak even though the strength of the mudstone layer is comparable to that of the siltstone and sandstone units. The mechanical properties of the rock are presented in Table 3.1. Overcoring and hydro fracturing stress measurements have been undertaken in Prince Mine and the data are summarized in Table 3.2.

Similar to the Lingan Mine, the main roadway intersections at Prince Mine are located in the vicinity of the main slopes and entries to the production panels. The current development plan at the Prince Mine is to reduce the number of intersections. In general, the layout of the three and four way intersections at Prince Mines are such that the roadways intersect at 30° to 40°.

3.2.1 Support for Main Roadways and Intersections

The main roadways at Prince Mine are excavated by Dosco Mark 2A and 2B roadheaders and supported by steel sets consisting of 80 x 100 mm x 21 kg/m arched sections spaced at 1 m centres. Timber lagging is used to block the steel set at the roof arch and on the sides. Steel corrugated sheets of 1.2 m x 2 m are installed behind the sets to cover the arch and the leg portions.

At the intersections, square work support is typically used. Figures 3.5 and 3.6 show the general arrangement of the structural members at the three and four way intersections. The sizes and properties of the structural members are summarized in Table 3.3. The square work support was installed in the slope prior to excavation of the cross cut.

3.2.2 Support Performance

Based on the information provided by the operations personnel at the mine, there has been very few problems with regard to the stability of the main slopes and the roadway intersections. Where a thin layer of mudstone is encountered at the roof, it often tends to delaminate at the contact of the overlying sandstone layer. However, conditions could be quite variable due to the presence of Paleo-river sandstone channels. Interbedding in the sandstone often gave rise to rock loosening at the roof. Nevertheless, the observed thickness of loosened rock at the roof is generally less than 0.6 m. No noticeable deflections of the steel arches and square work at the intersections were observed.

3.3 Phalen Mine

Phalen Mine is located at New Waterford adjacent to the Lingan Mine. The mining development area extends approximately 1.3 km offshore in a northerly direction. The workable coal seam at the mine is the Phalen seam which is about 1.8 m thick on the west side and about 2.5 m to 3 m thick on the east side of the mine. The coal seam is currently mined by retreat longwall mining techniques.

In general, the coal seam is overlain by a successive sequence of mudstone, siltstone and sandstone. Delamination tends to occur between the sandstone and mudstone. The presence of paleo-river sandstone channels often caused weak roof conditions. The mechanical properties of the rock are presented in Table 3.1. Three and four way intersections used at the Phalen Mine are located in the vicinity of the main slopes and entries to the production panels.

3.3.1 Support For Main Roadways and Intersections

The main roadways at Phalen Mine are excavated by Dosco and Anderson Strathclyde roadheaders and supported by steel sets of 80 x 100 mm x 21 kg/m arched sections spaced at 1 m centres. Timber lagging and steel corrugated sheets similar to those for the Lingan and Prince Mines are used with the steel sets.

Similar to Lingan and Prince Mines, square work supports are used at the roadway intersections. Figures 3.7 and 3.8 show the general arrangement of the structural members at the three and four-way intersections. The sizes and properties of the structural members are summarized in Table 3.3. The support for the intersection is installed in the main slope prior to the excavation of the cross cut.

3.3.2 Support Performance

According to the information provided by the operations personnel at the mine, the roof of roadways and their intersections are generally located in siltstone and the condition is generally stable and the steel support at the intersections only shows minor deflections.

An extensometer has been installed by the Cape Breton Coal Research Laboratory of CANMET at the intersection of 2 Angle Deep and 3 West Bottom. The extensometer was located between the third and fourth roof beams (spaced at 1 m centres), 2.6 m from the east side leg of the supporting steel frame. At the time of installation, the face was about 2.5 m ahead of the extensometer location, and the support was placed at about 3.0 m from the face.

The reported results are shown in Figure 3.9, which is a plot of the relative displacements of the anchors with respect to the reference surface anchor at the collar of extensometer hole (located at about 50 mm from the roof). The results indicate that the majority of the movements (90%) occurred within the first 2 m of the surface. Some minor delamination was noted between the 2 and 3 m anchors. The displacement pattern also indicated that the displacements increased with advance of the face indicating the effects of the gradual reduction of the restraints from the face and the continuing deformation of the rock despite the presence of the supporting roof beams. Such rock movements have apparently displaced the extensometer guide to such an extent that further reading was not possible after 4.5 m of advance. This was likely to be attributed to substantial loosening of the rock in the zone close to the roof, a result of the passive nature of the installed support system.

3.4 Current Design Practice of Support System for Main Roadway Intersections in the Sydney Coalfield

Based on our discussion with the Civil Engineering Department of CBDC, it is noted that the current structural design practice of the support for the roadway and roadway intersections has been based on an assumed rock load on the support. The rock load is generally assumed to come from a triangular distribution of loosened rock with the height of the triangle equal to half of the span of the opening under consideration. Figure 3.10 presents the assumed configurations of the rock loads for the arches and for roadway intersections. The derivation of the design rock load is apparently independent of the geological conditions encountered at the intersections.

The roof beams at the square work are designed as simply supported beams resting on collar beams which in turn are supported by columns. In some cases, the roof beams are directly supported by columns (e.g. along the wall of the main slope in a three-way intersection). Where the collar beams are connected to columns, a rigid moment connection is generally used to stiffen the joint. The collar beam is still however, designed as a simply supported beam.

The design stress for the structural members is equal to 100 percent of the yield stress (300 MPa) of the steel to offset the conservatism in the assumed rock load and end conditions of the beams. The current trend in the Lingan and Phalen Mines is to use a heavier collar beam in order to reduce the deflection of the beam as observed. Box girders for both mines as shown in Figure 3.3 are being proposed. This design philosophy tends to use the worst possible condition and disregards the rock conditions and the need to restrict the progressive loosening of the rock.

A review of the current design practice is made in the next section by a series of geomechanics analysis of the condition.

TABLE 3.1

MECHANICAL PROPERTIES OF ROCK IN THE LINGAN, PRINCE AND PHALEN MINES

	Unconfined Compressive Strength (MPa)	Young Modulus (MPa)	Poisson's Ratio	Tensile Strength (MPa)
LINGAN MINE				
Coal	11.0	3450	0.1	-
Mudstone	20.6	15450	0.11	-
Siltstone	48.3	36550	0.20	-
Sandstone	79.0	30340	0.18	-
PRINCE MINE				
Mudstone	20	20340	-	-
Siltstone	48.3	15800	0.16	-
Sandstone	41.3-48.3	17650	0.20	4.0
PHALEN MINE				
Mudstone	28	12700	-	3.4
Siltstone	50.6	16300	-	6.8
Sandstone	58.1	16200	-	5.5

TABLE 3.2

STRESSES MEASURED AT DIFFERENT MINES IN THE SYDNEY COALFIELD

Mine Site	Depth of Test Site (m)[1]	Method of Stress Measurement	Principal Stresses[2] (MPa) (Azimuth/Dip) (Degrees) σ_1	σ_2	σ_3	Stress Components[3] (MPa) σ_{E-W}	σ_{N-S}	σ_v	Overburden Stress (MPa)
Donkin Mine	165 (12)	overcoring	-	-	-	9.04	7.93	4.90	4.2
No. 26 Colliery	762 (90)	overcoring	26.3 (278/64)	24.05 (9/13)	23.2 (270/-25)	23.31	24.98	25.27	21.7
Prince	222 (20)	overcoring	11.8 (263/14)	9.54 (-7.8/5)	5.1 (64/-75)	10.76	9.51	6.00	5.5
		hydrofracturing	17.8-25.4 (274/0)	12.8-14.1 (6/0)	5.4-5.9 (-)	-	-	-	-
Lingan	593 (70)	overcoring	33.84 (88/4.0)	14.49 (4/-55)	12.21 (355/35)	33.73	13.76	13.05	14.3
		hydrofracturing	-	-	7.8/15.4 (-)	-	-	-	-

Notes:

(1) Depth below sea level, Figure in ( ) indicates depth of water.

(2) σ_1, σ_2, σ_3: Major, intermediate and minor principal stresses.

(3) σ_{E-W}, σ_{N-S}, σ_v: Stress components in E-W, N-S and vertical directions.

June 1988

871-1369

TABLE 3.3

PROPERTIES OF STRUCTURAL MEMBERS
USED FOR INTERSECTION SUPPORT OF LINGAN, PRINCE AND PHALEN MINES

	Description Metric (Imperial)	Nominal Mass kg/m	Ix $\times 10^{-6}$ (m^4)	Sx $\times 10^{-6}$ (m^3)	Depth (mm)	Fy (MPa)	Fu (MPa)	Utilisation (year)
LINGAN MINE								
Collar Beam	W200 x 59 (W8 x 40)	59	61.1	582	210	300	450-620	4 way-design (76)
	W310 x 107 (W12 x 72)	107	248	1590	311	300	450-620	4 way-design (86)
	W310 x 179 (W12 x 120)	179	445	2680	333	290	450-620	4 way-design (86)
	BOX GIRDER	-	567	2836	400	300	450-620	4 way-design (80)
	W310 x 202 (W12 x 136)	202	520	3050	311	290	450-620	3 way-design (86)
Roof Beam	W200 x 52 (W8 x 35)	52	52.7	512	206	300	450-620	4 way-design (76 to 88)
	W250 x 80 (W10 x 54)	80	126	982	256	300	450-620	3 way-design (86) 4 way-design (88)
PRINCE MINE								
Collar Beam	W310 x 74 (W12 x 50)	79	177	1160	306	400	520-690	4 way and 3 way intersection
	W310 x 107 (W310 x 72)	107	248	1590	311	300	450-620	4 way-design (87)
	W310 x 118	118	275	1750	314	300	450-620	3 way-design (87)
Roof Beam	W200 x 59 (W8 x 40)	59	61.1	582	210	400	520-690	
PHALEN MINE								
Collar Beam	BOX GIRDER	-	567	2836	400	300	450-620	4 way-design (88)
	W310 x 107 (W12 x 72)	107	248	1590	311	300	450-620	4 way-design (86)
Roof Beam	W310 x 79 (W12 x 53)	79	177	1160	306	300	450-620	4 way-design (86) 3 way-design (85)

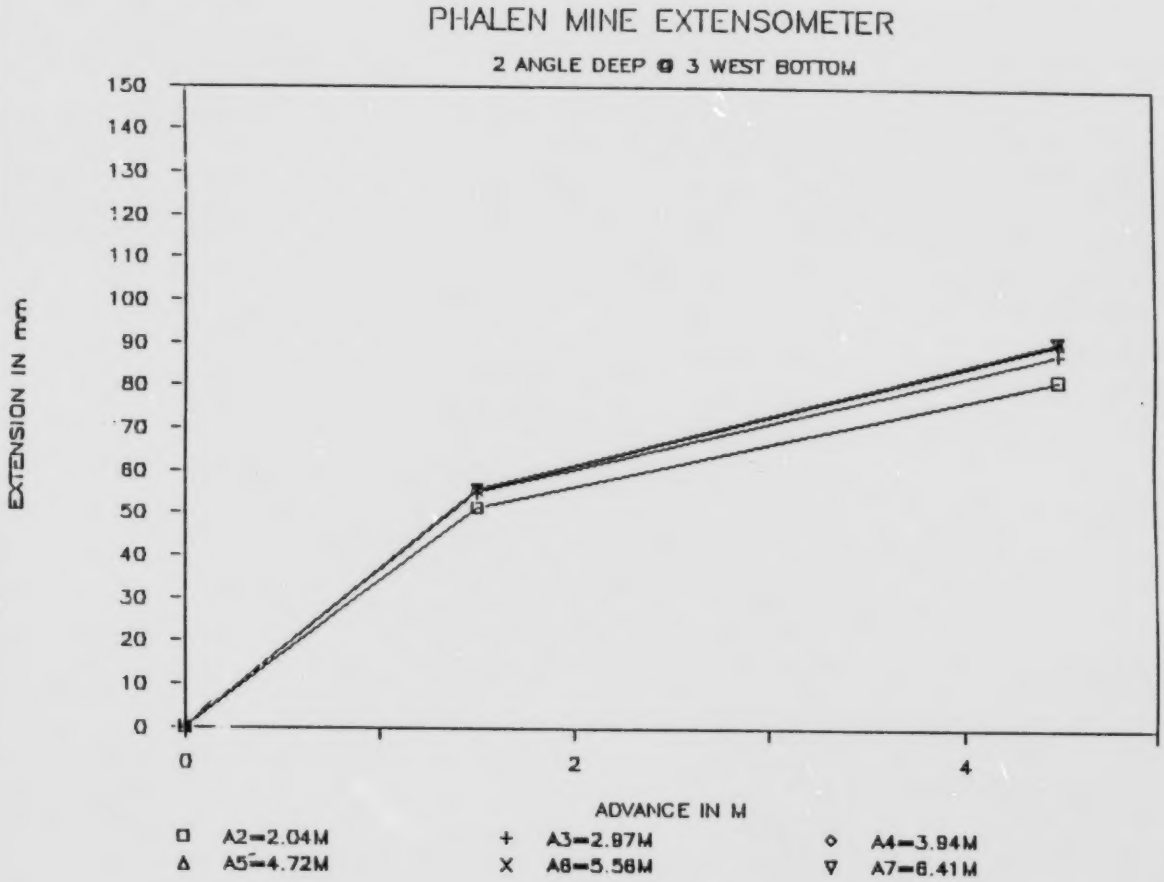
Notes: Young Modulus (E) = 206 GPa of Steel
Ix - Moment of inertia about the x-x axis
Sx - Modulus of section about the x-x axis
Fy - Yield stress of steel
Fu - Ultimate stress of steel
(Data provided by CBDC)

June 1988

871-1369

PLOT OF EXTENSOMETER RESULTS (PHALEN MINE)

FIGURE 3.9



REFERENCE:

DATA AVAILABLE FROM CANMET CRL (ASTON 1988)

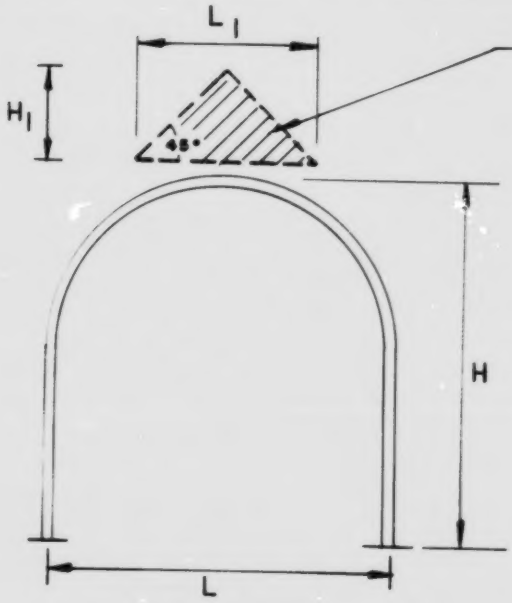
Date MAY 26,1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd F.M.

ASSUMED CONFIGURATION OF THE ROCK LOADS FOR THE DESIGN OF ARCHES AND INTERSECTION SUPPORT CBDC DESIGN PRACTICE

FIGURE 3.10



ASSUMED TRIANGULAR LOAD DISTRIBUTION OVER THE ARCH APEX

WHERE:

H = ARCH HEIGHT (m)

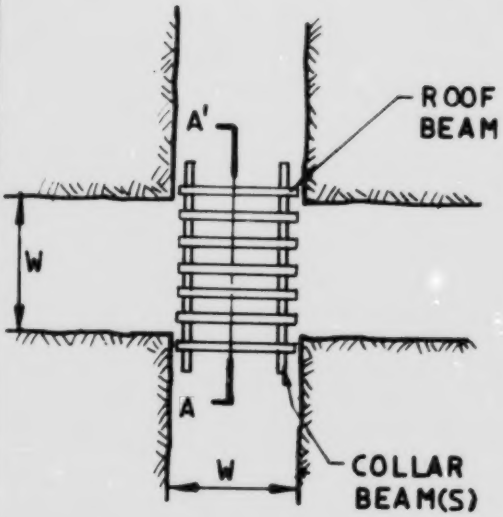
L = ARCH WIDTH (m)

L_1 = ASSUMED WIDTH (m)

$H_1 = L_1 / 2$ (m)

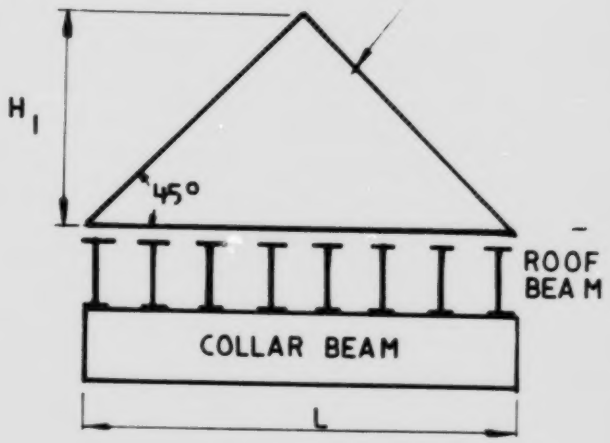
$\gamma = 27000\ N/m^3$

STEEL ARCH



SKETCH OF TYPICAL STEEL SUPPORT FOR INTERSECTION

ASSUMED TRIANGULAR LOAD DISTRIBUTION OVER THE COLLAR BEAM



SECTION A-A'

WHERE:

L = COLLAR BEAM LENGTH (m)

$H_1 = L/2$ (m)

W = WIDTH (m)

INTERSECTION SUPPORT

NOT DRAWN TO SCALE

Date MAY 30, 1988

Project 871-1389

Golder Associates

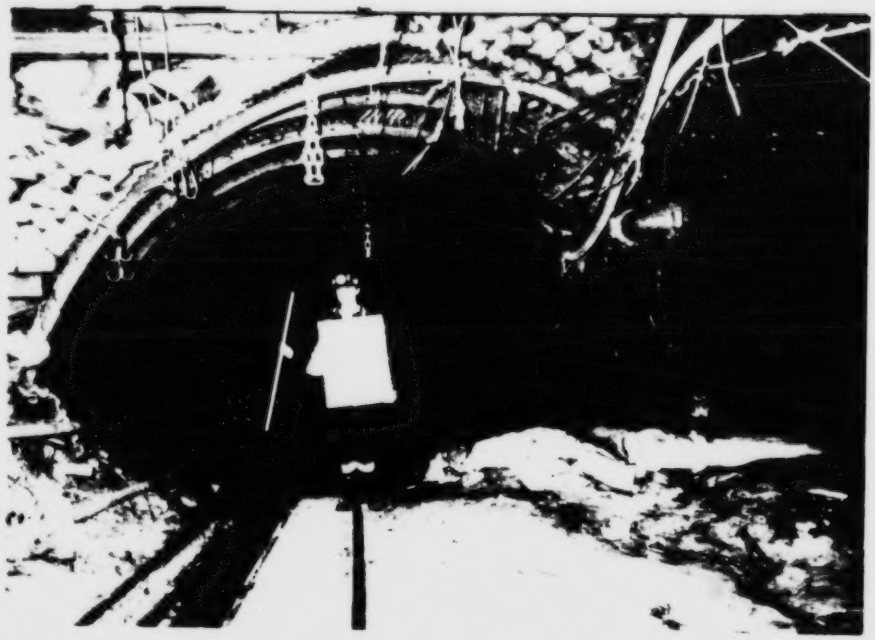
Drawn D.M.

Chkd F.M.

Form 0.A.-0-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 1, FIG. 3.4)
4 SLOPE , 2 EAST BOTTOM

PLATE 1A



LOOKING NORTH (DOWN)

LOOKING SOUTH (UP)

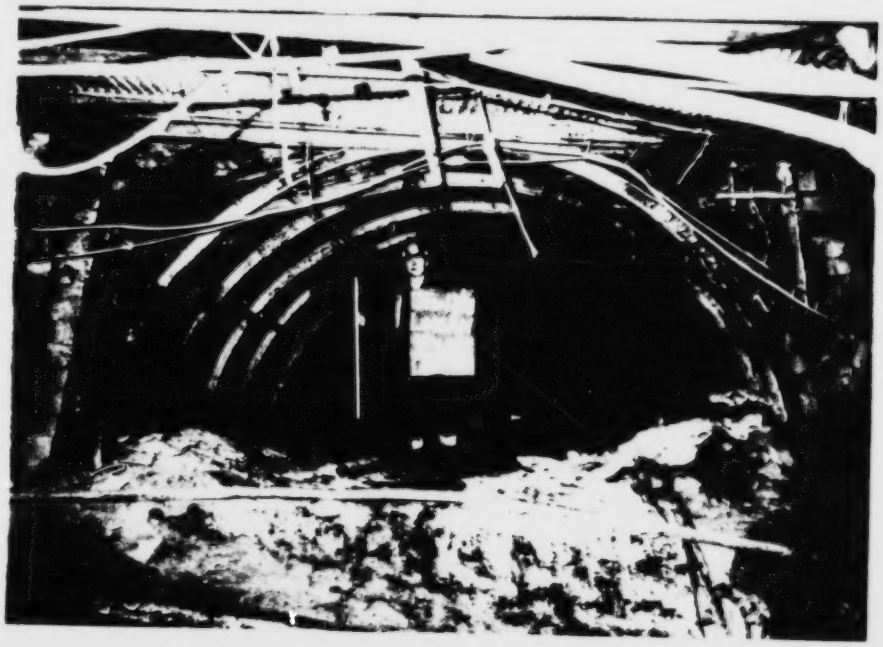
Date APRIL 12, 1988
Project 871-1369

Golder Associates

Drawn D.M.
Chkd F.M.

LINGAN MINE - 4 WAY INTERSECTION (POINT 1, FIG. 3.4)
4 SLOPE , 2 EAST BOTTOM

PLATE 1B



LOOKING WEST

LOOKING EAST

Date APRIL 12, 1988
Project 871-1369

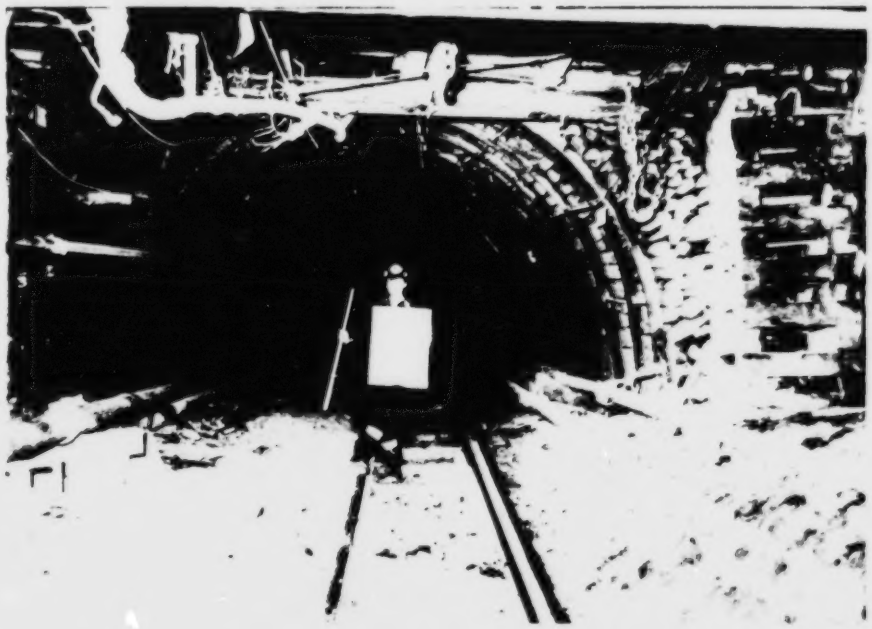
Golder Associates

Drawn D.M.
Chkd P.M

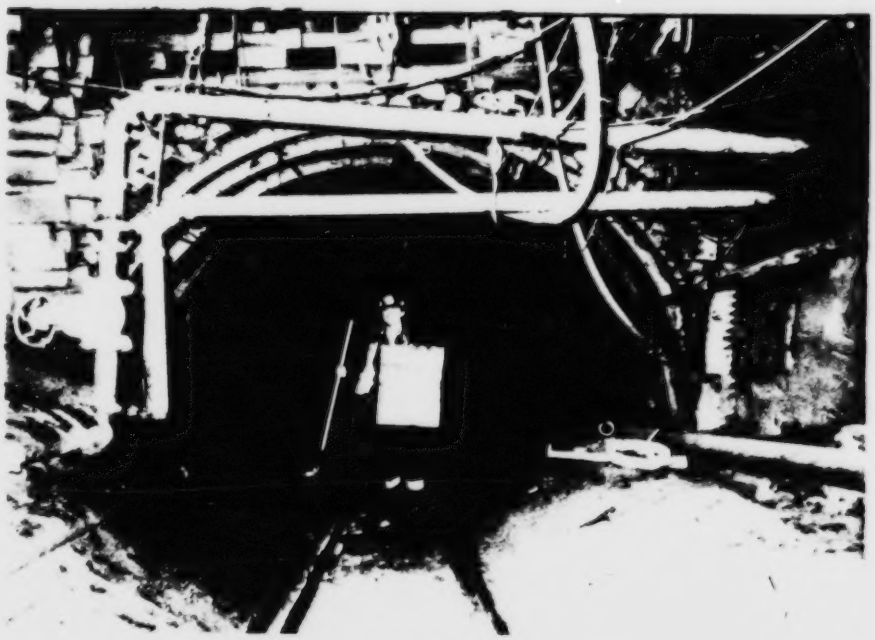
Form 6A - Dec/Imetric

LINGAN MINE - 4 WAY INTERSECTION (POINT 2, FIG. 3.4)
4 SLOPE, 3 EAST BOTTOM

PLATE 2A



LOOKING NORTH (DOWN)



LOOKING SOUTH (UP)

Date APRIL 12, 1988
Project 871-1369

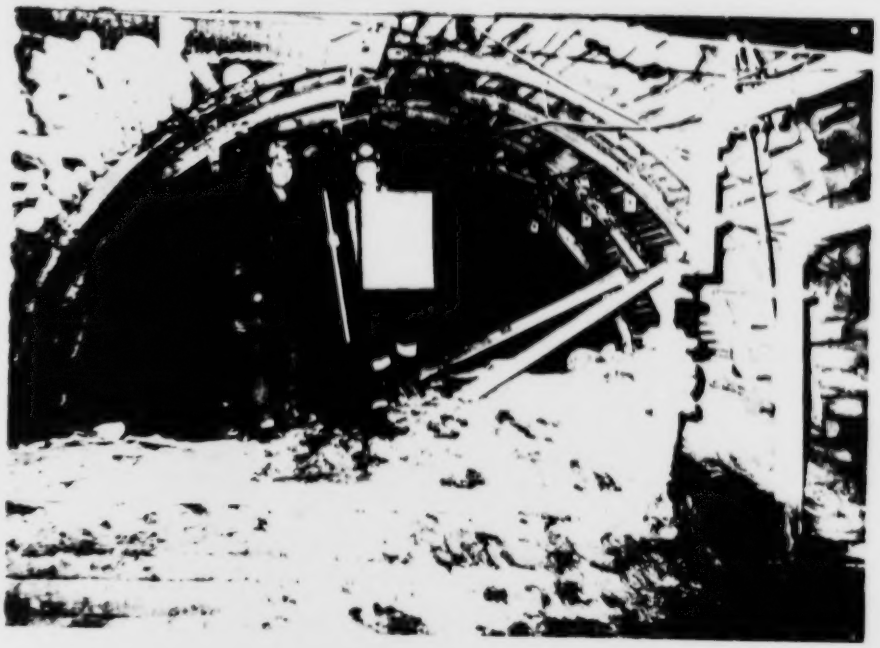
Golder Associates

Drawn D.M.
Chkd P.M.

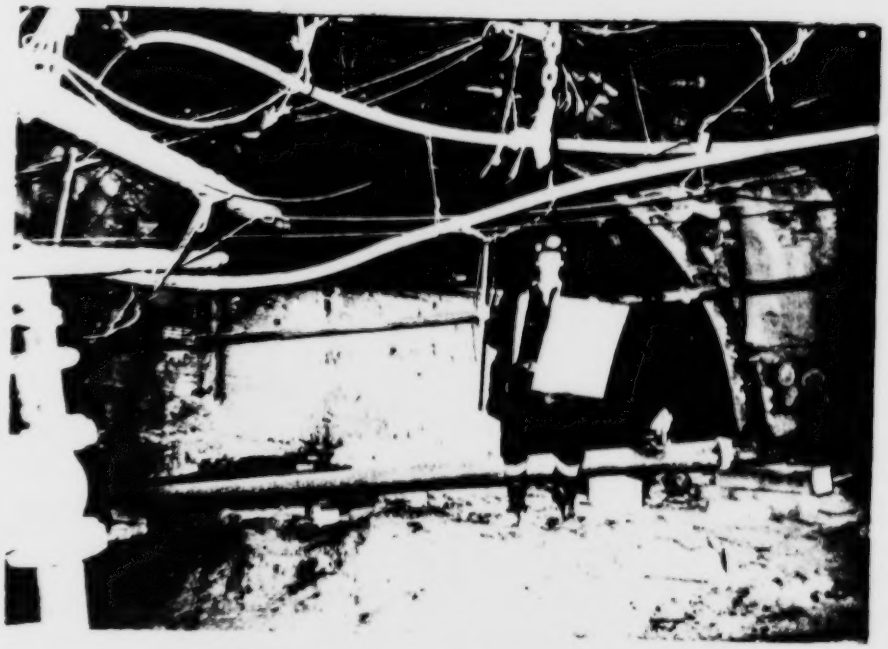
Form GA-D-4m (metric)

LIN GAN MINE - 4 WAY INTERSECTION (POINT 2, FIG. 3.4)
4 SLOPE, 3 EAST BOTTOM

PLATE 2 B



LOOKING WEST



LOOKING EAST

Date APRIL 12, 1988
Project 871 - 1369

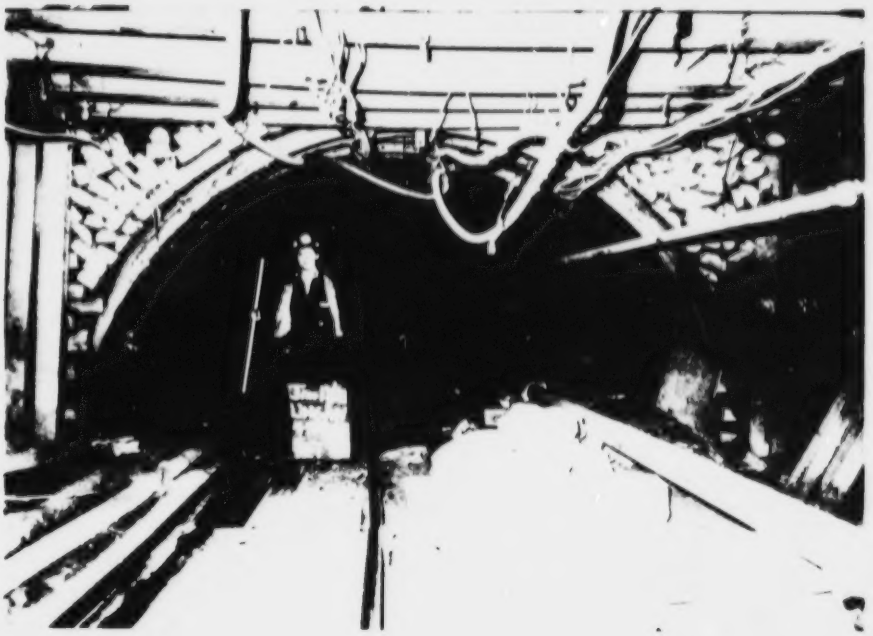
Golder Associates

Drawn D.M.
Chkd F.M.

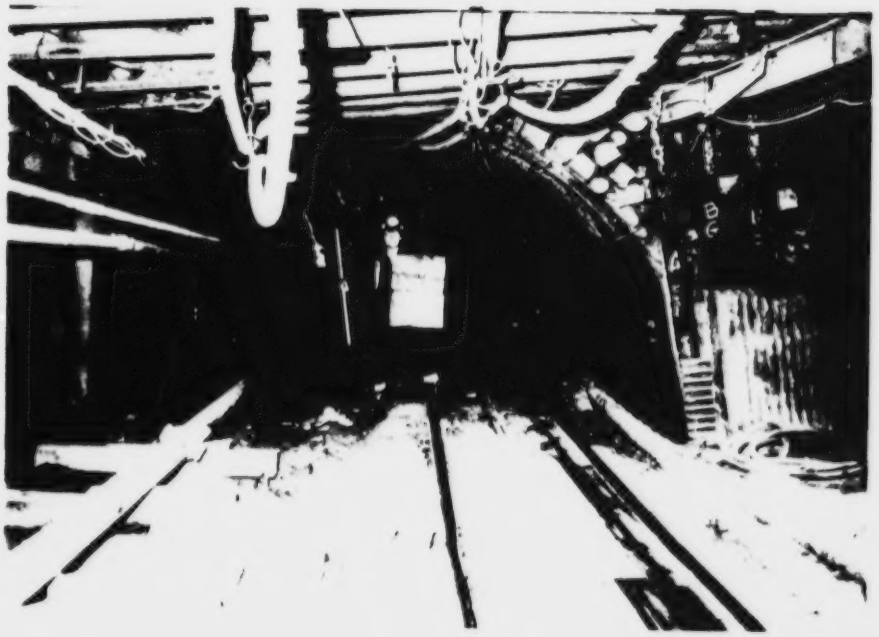
Form G.A.-D-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 3, FIG. 3.4)
4 SLOPE, 4 EAST TOP

PLATE 3A



LOOKING NORTH (DOWN)



LOOKING SOUTH (UP)

Date APRIL 12, 1988
Project 871-1369

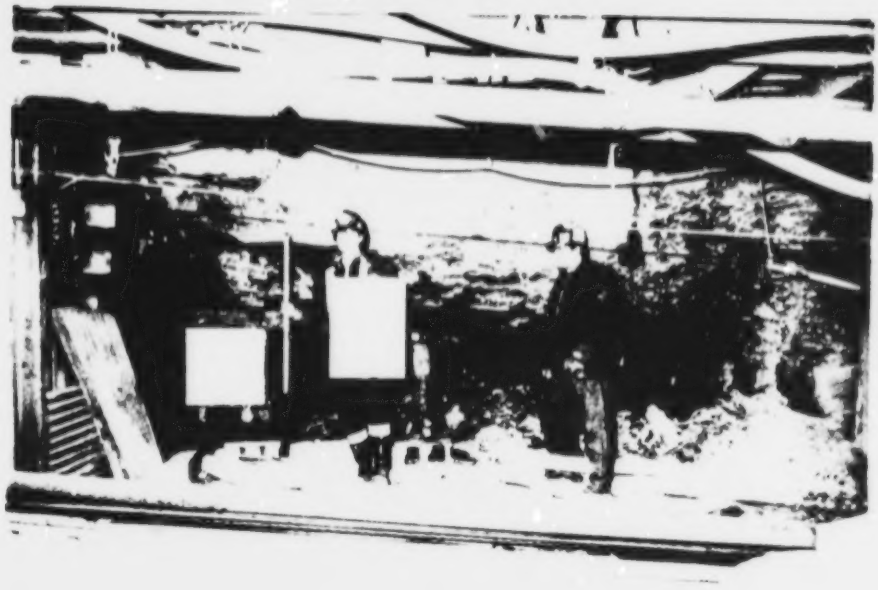
Golder Associates

Drawn D.M.
Chkd F.M.

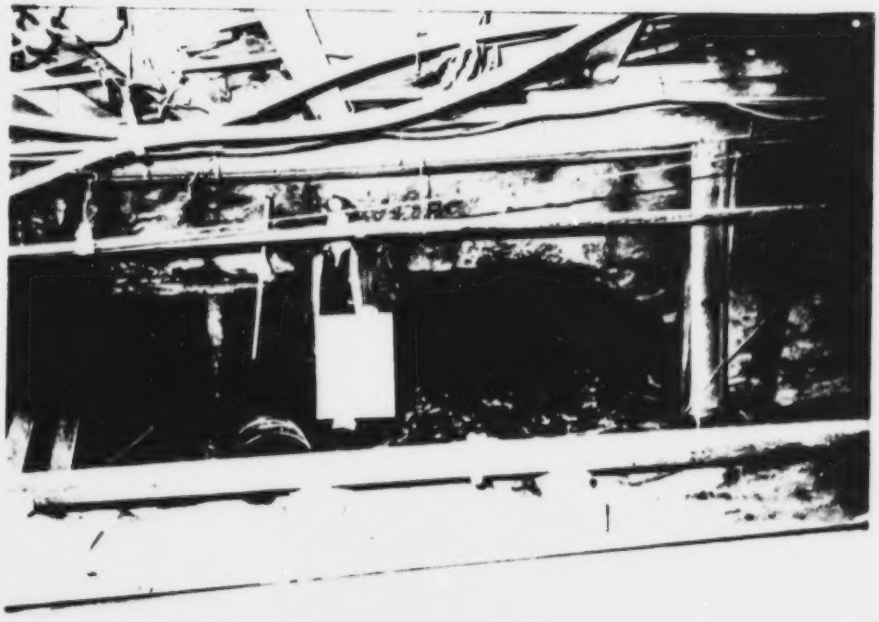
Form GA-D-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 3, FIG. 3.4)
4 SLOPE, 4 EAST TOP

PLATE 3.B



LOOKING WEST



LOOKING EAST

Date APRIL 12, 1988
Project 871-1369

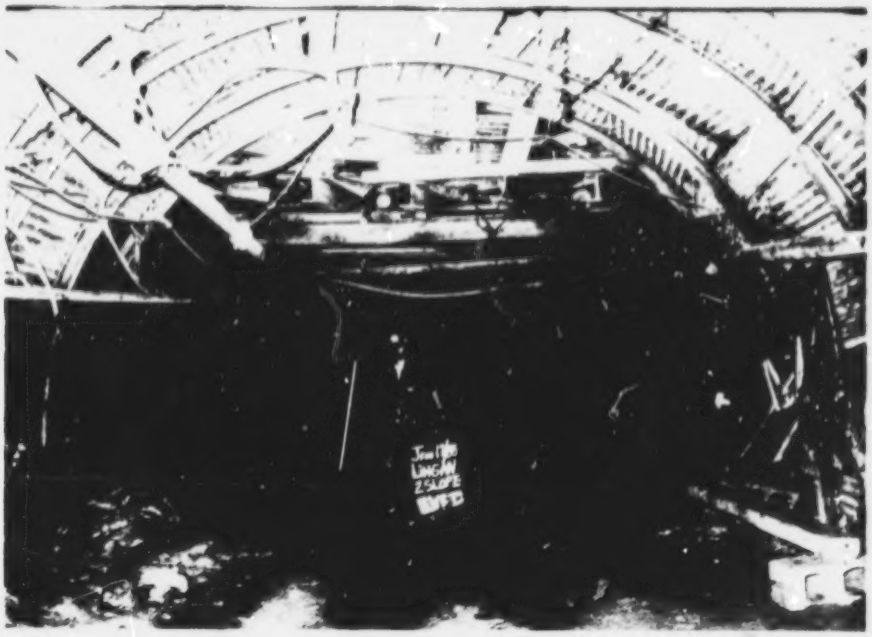
Golder Associates

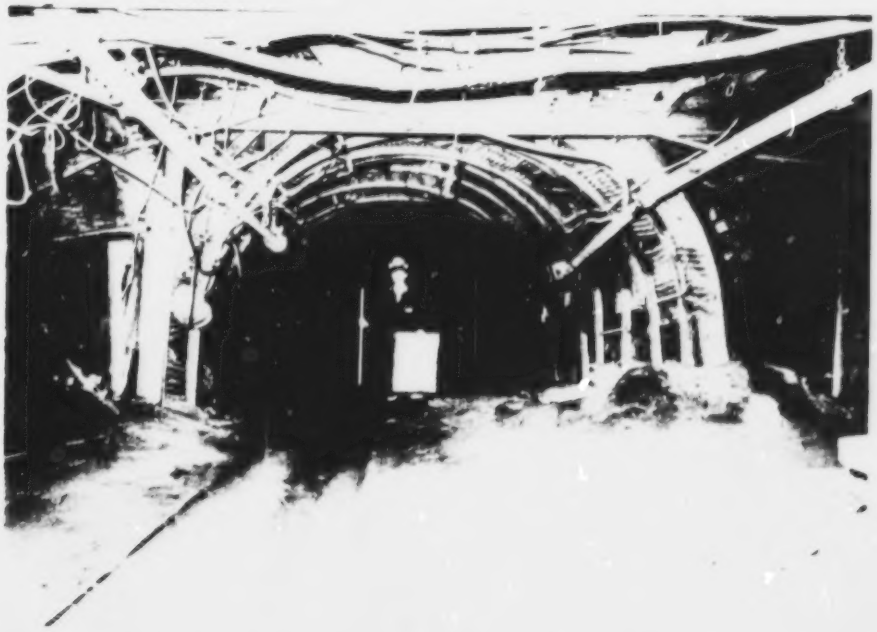
Drawn D.M.
Chkd F.M.

Form GA-D-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 4, FIG. 3.4)
2 SLOPE, 5 WEST BOTTOM

PLATE 4





LOOKING NORTH (DOWN)

Date APRIL 12, 1988
Project 871 - 1369

Golder Associates

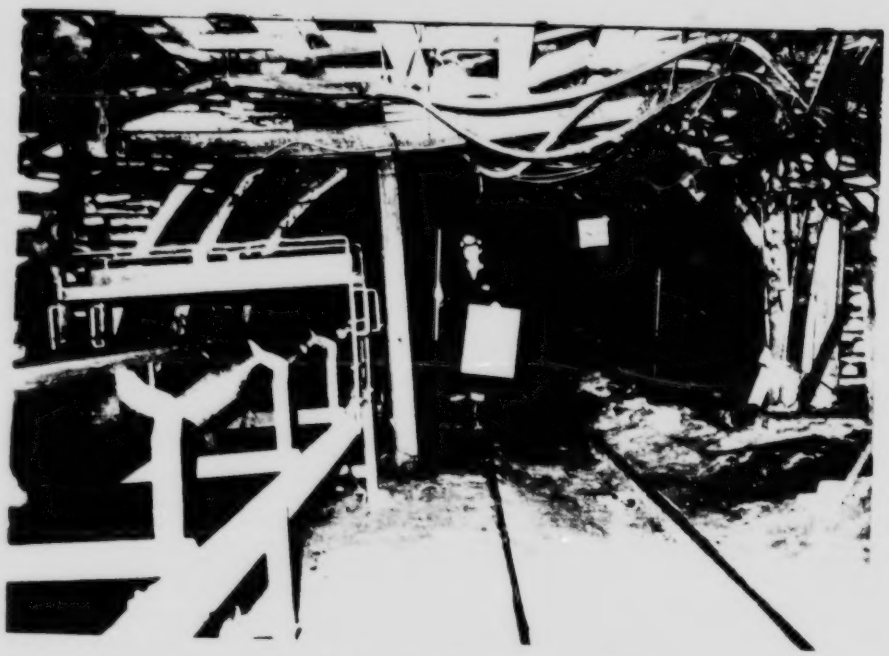
Drawn D.M.
Chkd F.M.

Form GA-D-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 5, FIG. 3.4)
3 SLOPE 10 EAST TOP

PLATE 5.A

LOOKING NORTH (DOWN)



LOOKING SOUTH (UP)

Date APRIL 12, 1988
Project 371-1369

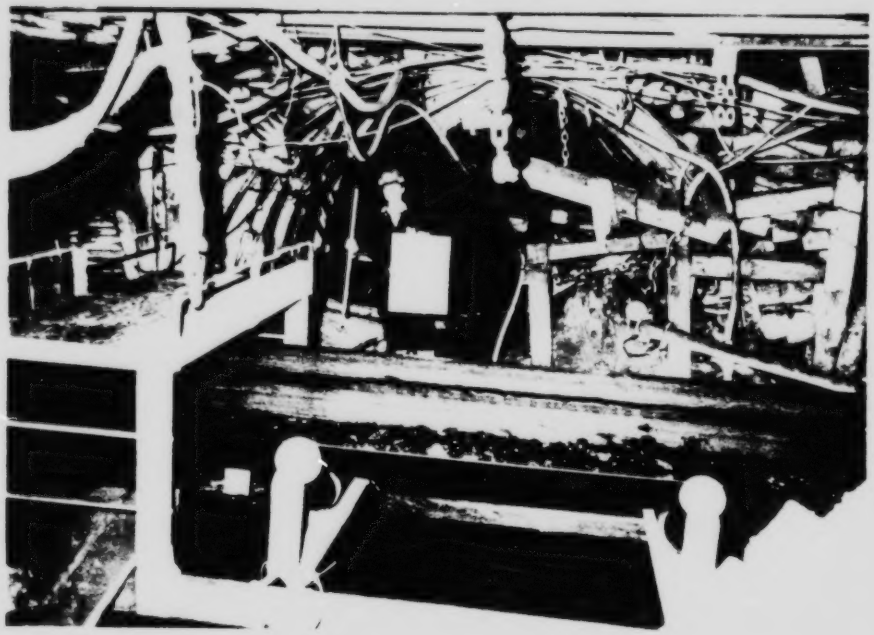
Golder Associates

Drawn D. M
Chkd S.M.

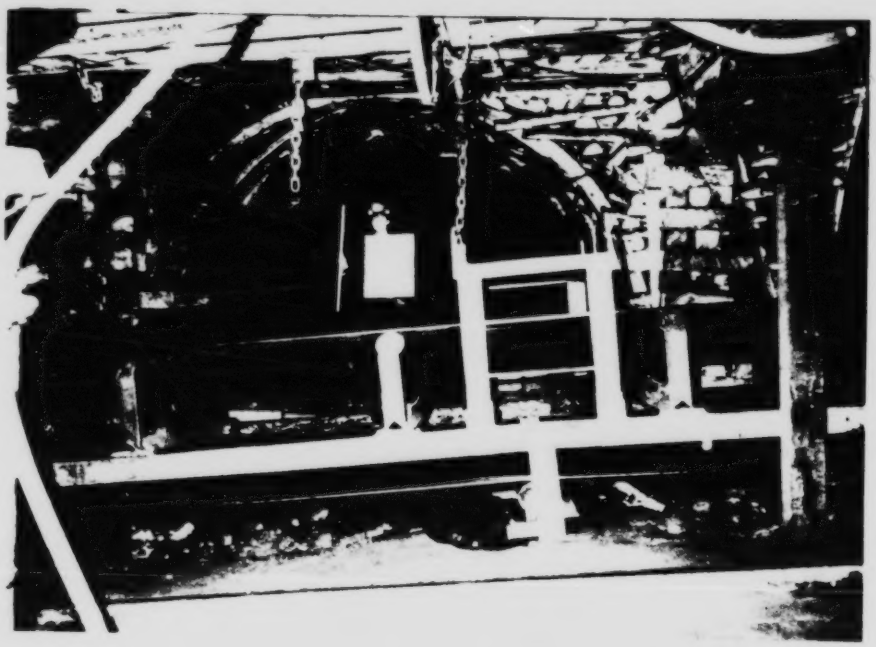
Form GA-D-4m (metric)

LINGAN MINE - 4 WAY INTERSECTION (POINT 5, FIG.3.4)
3 SLOPE, 10 EAST TOP

PLATE 5.B



LOOKING WEST



LOOKING EAST

Date APRIL 12, 1988
Project 871-1369

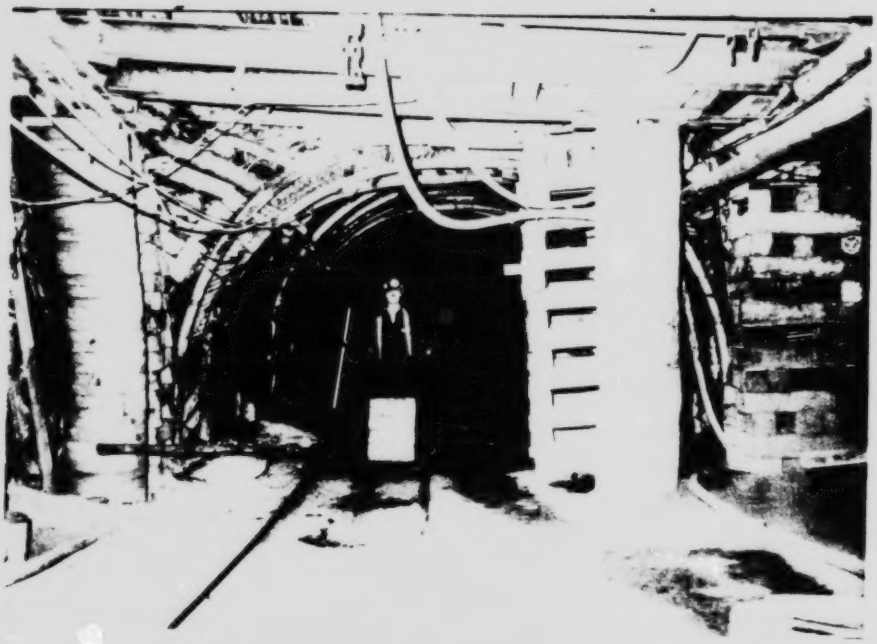
Golder Associates

Drawn D.M.
Chkd F.M

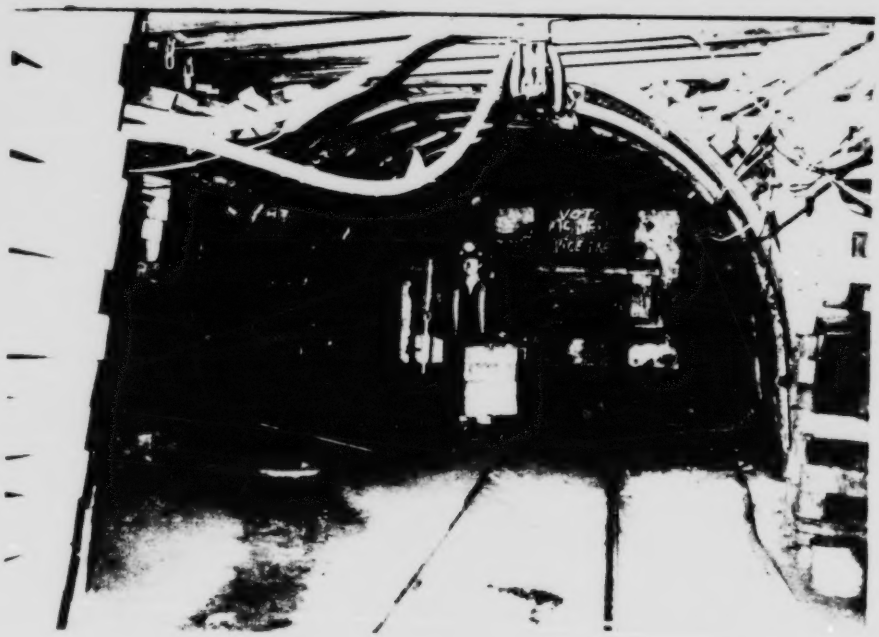
Form G.A.-D-4m (metric)

LINGAN MINE - 3 WAY INTERSECTION (POINT 6, FIG. 3.4)
2 SLOPE, 9 EAST TOP

PLATE 6.A



LOOKING NORTH (DOWN)



LOOKING SOUTH (UP)

Date APRIL 12, 1988
Project 871-1369

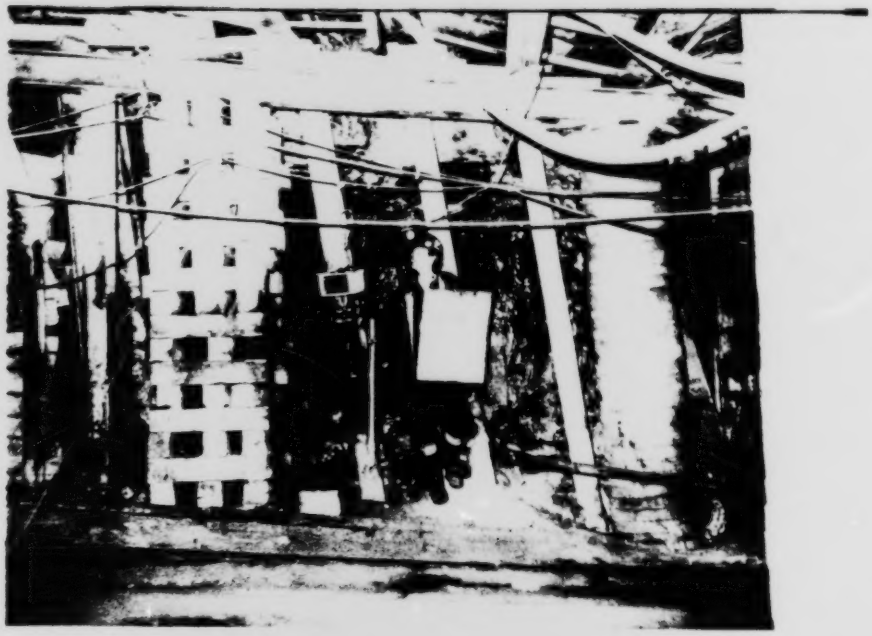
Golder Associates

Drawn D.M.
Chkd

Form G A - D - 4m (metric)

LINGAN MINE - 3 WAY INTERSECTION (POINT 6, FIG. 3.4) 2 SLOPE , 9 EAST TOP

PLATE 6.B



LOOKING WEST

LOOKING EAST

Date APRIL 12, 1988
Project 871 - 1369

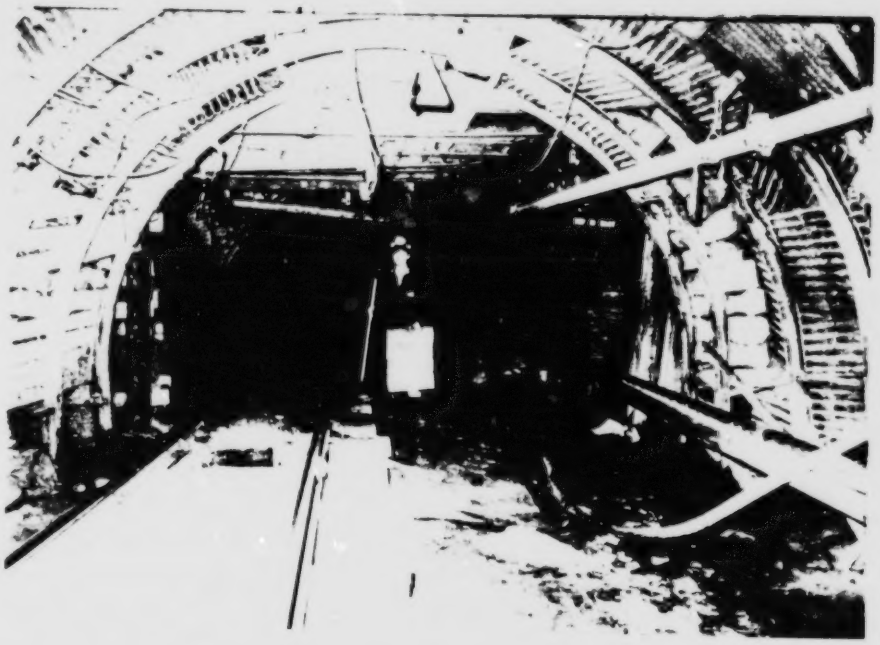
Golder Associates

Drawn D. M.
Chkd F.M.

Form G.A.-D-4m (metric)

LINGAN MINE - 3 WAY INTERSECTION (POINT 7, FIG. 3.4)
2 SLOPE , 10 EAST TOP

PLATE 7



LOOKING NORTH (DOWN)

LOOKING EAST

Date APRIL 12, 1988
Project 871-1369

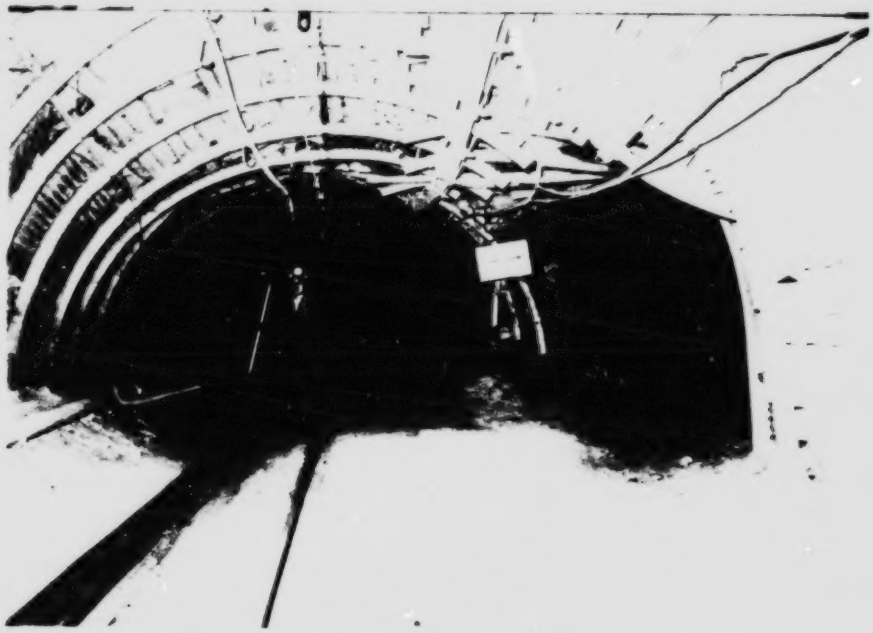
Golder Associates

Drawn D. M.
Chkd F. M

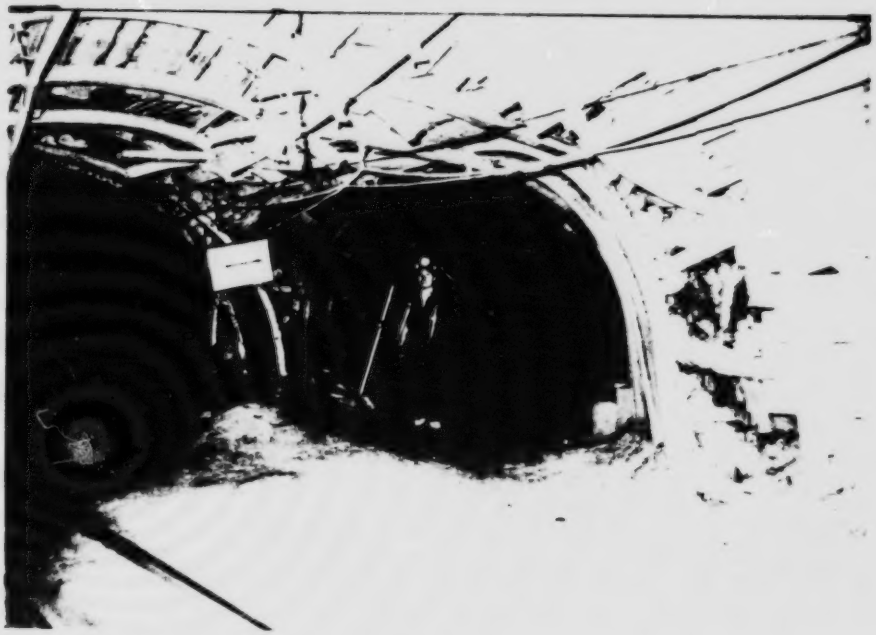
Form GA-D-4m (metric)

LINGAN MINE - 3 WAY "Y" INTERSECTION (POINT 8, FIG. 3.4)
4 SLOPE, 6 EAST BOTTOM

PLATE 8



LOOKING NORTH (DOWN)



LOOKING EAST

Date APRIL 12, 1988
Project 871-1369

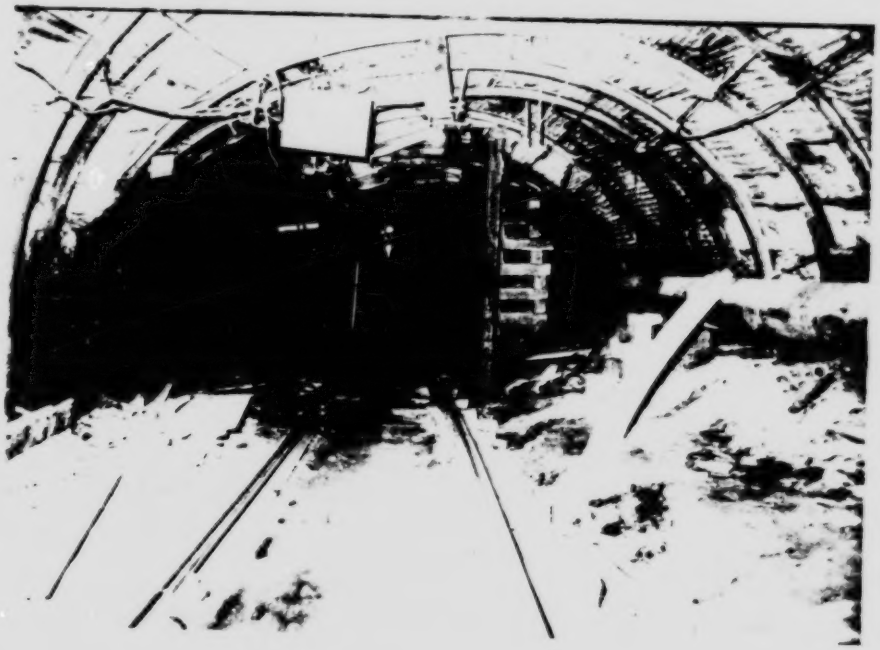
Golder Associates

Drawn D.M.
Chkd F.M.

Form GA-D-4m (metric)

LINGAN MINE - 3 WAY "Y" INTERSECTION (POINT 9 FIG. 3.4)
2 SLOPE, 4 WEST BOTTOM

PLATE 9



LOOKING NORTH (DOWN)

LOOKING WEST

Date APRIL 12, 1988
Project 871-1369

Golder Associates

Drawn C. M.
Chkd F.M.

Form GA-D-4m (metric)

4. REVIEW AND EVALUATION OF MAIN ROADWAY INTERSECTIONS FOR THE LINGAN MINE

For the purpose of this study, the Lingan Mine was selected as the reference mine for the detailed review and evaluation of the main roadway intersection design practice in the Sydney Coalfield.

As described in the previous section ,the current support systems for the main roadway and roadway intersections comprise steel arches and steel frames respectively. These roadways are generally protected by barrier pillars which separate them from the longwall panels. Observations made in the mine and discussions with the mine operations personnel indicated that there had been relatively minor deflections in the arches along the main slopes in comparison with the relatively large deflections observed in the arches along the gate roads. It appears that the rock loads or pressures on the arches result primarily from the localized failure of the rock surrounding the roadway due to the creation of the underground opening rather than induced by rock fracturing arising from longwall extraction.

Similarly, the rock loads at the roadway intersections would be predominantly influenced by the rock conditions, the insitu stresses and the support used.

The steel set support currently used for the slopes and the intersections are considered as a passive support system. This type of support allows rock deformation to take place while the pressure on the support system is gradually increased until an equilibrium condition is reached at which the rock deformation and support pressure are stabilized.

Figure 4.1 illustrates schematically the load deformation curves for the rock mass and the support system for a tunnel excavated by full-face drill and blast method with blocked steel sets being installed after each mucking cycle. Hydrostatic in situ stresses were assumed in the example. It may be noted that the rock will undergo elastic deformation and in-elastic deformation resulting from localized overstressing and rock loosening prior to installation of the support system. Rock deformation will continue as the support pressure is gradually built up to restrict further rock loosening and hence deformation. At equilibrium, the support will sustain external pressure from the rock represented by points E and F in Figure 4.1.

The same concept of rock support interaction applies to the roadway intersections. At present, there has not been any direct measurement (in Lingan Mine) of the depth of loosened zone above the roadway intersection nor the strains in any of the structural members. The only semi-quantitative data relating to the support performance are the estimated deflections of the roof and collar beams and the exposed condition at the brows at some of the intersections. In general, the support systems at the intersection are stable. The observed beam deflections generally range from 50 to 70 mm with larger deflection at one location described in Section 3.1.2.

However, it is important to note that the design of the roadway intersection support at the mines was undertaken based on an empirical rock load concept described in Section 3.4. No consideration has been given to the rock conditions. For example, in one of the intersections, massive sandstone was encountered at the roof. Such condition may not require the use of heavy sets as installed.

While it is recognized that standardized designs are preferred to facilitate field implementation, they do not, however provide the flexibility to optimize or economize the support design.

In the following section, the performance of roadway intersections at the Lingan Mine is evaluated. The objective is to develop a rational design procedure which takes into account the geological conditions, the in situ stresses and the rock mass characteristics.

4.1 Evaluation of the Support System For Roadway Intersections

The general arrangement of the support system (designed in 1976) for a typical four-way intersection at the Lingan Mine is shown in Figure 3.2. The section properties of the structural members are summarized in Table 3.2.

Collar Beam

The collar beam is designed to support the roof beams which are spaced at 0.6 m centres (as shown in Figure 3.2). Due to the close spacing of the roof beams, the loading on the collar beam could be approximated by a uniformly distribution load equal to the total point loads from the roof beams. Using simple beam theory (see Appendix B), the loading on the collar beam could be back-analysed from the observed/estimated beam deflection.

As there were no direct field measurements of beam deflection, a range of values was considered. Table 4.1 summarizes the estimated equivalent uniformly distributed rock loads (expressed in terms of the height of loosened

rock per unit area over the intersection) on the collar beam corresponding to the deflections.

According to the results presented in Table 4.1, the collar beam would reach its yield stress of 300 MPa when a uniformly distributed load equivalent to the weight of 0.8 m of loosened rock is applied over the projected area of the structure at the intersection. The corresponding centre-span deflection of the beam under this loading condition is about 77 mm. On the basis of this calculation, the collar beams at most intersections probably have not reached the yield stress even though they are sustaining noticeable deflections of between 50 to 70 mm.

In conventional design of structural steel beams, the maximum deflection recommended by the Canadian Institute for Steel Work for simple beam under static load is 1/300 of span. This corresponds to about 20 mm for the maximum allowable deflection for the collar beam. Apparently this criterion was exceeded at most of the intersections. Nevertheless, in a mining environment larger beam deflections are being tolerated.

Roof Beams

For a given uniformly distributed loading on the collar beam, several loading cases for the roof beams are possible. The loading may be transmitted to the roof beam in the form of:

(a) uniformly distributed load
(b) point load at the mid span of the beam
(c) point loads at the mid and quarter points
(d) symmetrical triangular distributed load

These loading configurations are illustrated in Figure 4.2. For each loading case, the mid-span deflection and the maximum bending stress of the roof beam were evaluated for different equivalent thicknesses of loosened rock. The results are summarized in Table 4.2 and plotted in Figures 4.3 and 4.4.

Summary

The results obtained from the above analyses are summarized below.

(a) The observed/estimated deflection of the collar beam at the four-way intersection at Lingan is generally about 50 mm to 70 mm where siltstone is encountered at the roof. According to the back calculations presented in Table 4.1, the equivalent depth of loosened rock which would give rise to such deflection is about 0.6 m. The estimated total load on the support system would be about 680 kN.

(b) The fact that yielding (distortion) of the collar beam was not observed indicates that the yield stress in the beam has not been exceeded. The equivalent uniform thickness of loosened rock is therefore not likely to exceed about 0.8 m (see Table 4.1), or the total load over the support structure is not likely to exceed 907 kN.

(c) The rock load as discussed above could be imposed on the support system in a number of possible configurations as shown on Table 4.2. Under the average condition, the equivalent thickness of loosened rock is about 0.6 m (total load of 680 kN over the intersection support). The corresponding deflection of the roof beam, under different loading

configurations ranges from 18 to 29 mm. This appears to be consistent with the observed condition of the roof beams at the intersection - minor deflection with no indication of distortion.

A similar analysis of the typical support arrangement (Figure 3.1) for a 3-way intersection was undertaken. The results are presented in Tables 4.3 and 4.4 and plotted in Figures 4.5 and 4.6. The salient points are summarized below.

a) The collar beam proposed for the 3-way support is not likely to reach the yield point (290 MPa) for an estimated height of loosened rock of less than 1.5 m. The heavy section of the collar beam (see Table 3.2) allows a higher rock load to be supported.

b) The rock load could be imposed on the roof beam in a number of possible configurations as shown previously for the four-way intersection. The deflection calculated (Table 4.4) for different loading configurations are relatively small; in the range of 3 to 26 mm.

The results of the analyses for both the three and four-way intersections indicate that the rock loads on the support came directly from the loosened zone above the roof of the intersections. The extent of this zone is assessed by back calculating from the estimated deflection of the roof beams. The zones of loosening is influenced by:

- the geometry of the opening
- the rock condition of the roof including the in situ stress condition
- the support system used

Thus, a stability analysis which takes into account the above factors would be useful for assessing the rock loading at the intersection. The type of analysis which may be useful to address this aspect is discussed in the following section.

4.2 Stability Analysis of Roadway Intersections at Lingan Mine

Based on the available geological information and the reported conditions in the underground coal mines, the stability of the main roadways and intersections is predominantly governed by rock loosening due to excavation of the underground opening.

With reference to the Lingan Mine, the major discontinuities encountered in the mines are the bedding surfaces and the contacts between different stratigraphic units. There are generally very few transverse joint sets which would cause wedge failures in sidewalls of roadways and in the pillars at the intersections.

As far as groundwater effects are concerned, there has been very little groundwater problems encountered in the underground mines in the Sydney coalfield. In general, the mines are dry. Hence, the groundwater effects on the joint sets and rock mass strength are considered to be minimal.

Induced stresses are considered to be the dominant factor which has affected the stability at the main roadways and their intersections. As described earlier in Section 3, the main roadways are separated by barrier pillars of 65 m and they are also isolated from the advancing longwall panels (in Lingan Mine) by a minimum pillar of 80 m.

Observations made in the mines have indicated that the longwall extraction has little effect on the performance of the support systems at the roadway intersections. That is, there has not been reported deterioration of the intersection supports due to progressive extraction of the near-by longwall panels. Analyses carried out for rock and gas outburst studies for No. 26 Colliery have indicated that the longwall extraction would tend to reduce the tangential stress concentration at the roof of the main roadway because of stress sheltering effects (Yuen, 1987). The rib pillar between the longwall and the gate road supports however will experience an increase in rock loads with the extraction.

4.2.1 Stress Analysis

4.2.1.1 Analysis of Stresses and Deformations of Pillars at Intersections

It is recognized that the distribution of stresses and deformations occurring around an intersection is three dimensional. Stress analyses using three dimensional finite element code were carried out by Gercek (1982) to examine the stresses and deformations in the pillars for isolated intersections and for intersections in room and pillar mining for US coal mines (refer to review of research studies relating to intersection design in section 2).

For the purpose of this study, a three dimensional stress analysis was carried out using the NFOLD computer program to determine the normal stresses and deformations in the pillars around the intersection. The NFOLD program utilizes the displacement discontinuity method to determine stresses and displacements normal and parallel to a tabular ore body. It takes into account the different deformation characteristics of the coal seam and the surrounding strata.

Two reference intersections were analyses: a four-way and a three-way intersection as shown in Figures 4.7 and 4.8. The intersections were assumed to be located at a depth of about 650 m below sea level. The parameters used for the analyses are presented in Table 4.5. The mechanical properties of the rock were based on the data presented in Table 3.1 and the in situ stresses were based on the in situ stress data in Golder Associates Report (851-6025). The extent of the model was chosen with reference to the results of the three dimensional stress analysis performed by Gercek (1982) for three and four-way intersections. Elastic analyses were carried out for the two intersections.

Results of Analysis

Figure 4.9 shows the elastic convergence of the roadways and cross cuts for the three-way intersections. The normal stresses in the pillars and abutment in the vicinity of the intersections are shown in Figure 4.11. The corresponding convergence and normal stress distributions for the four-way intersections are presented in figures 4.10 and 4.12 respectively. The salient points from the results are as follows:

- The elastic convergence for the 4-way intersection is marginally higher than that for the three-way intersection as expected. It is noted that the maximum calculated elastic convergence is about 13 mm. This magnitude of movement represents the instantaneous response of the rock to the excavation of the intersections and will be substantially less than the observed or measured displacement of the roof strata which reflects the cumulative effects of progressive rock loosening.

- Based on the convergence pattern the influence zone of the roadway extends into the main roadway and cross cuts for a distance at least equal to the width of the intersecting roadways.

- The effect of the intersection development is to create stress concentrations in the abutments/pillars adjacent to the intersection. The normal stress is highest at the corners and gradually reduces to the ambient (in situ) values. The area of major influence extends to about 3 times the width of the roadway.

- The model predicts an average stress concentration of 1.5 at the corner and about 1.3 at the abutment zone. Similar trends are noted in both three and four-way intersections. On the basis of the stress condition shown in Figures 4.11 and 4.12 and the rock strength in Table 4.5, overstressing of the corner of the intersection is anticipated. Further into the pillar, the rock is subject to higher confining stresses and the risk of overstressing would be reduced. This effect would be better assessed if the triaxial strength of the coal is available.

- Based on field observations, the corners of the intersections were trimmed off to make way for the installation of the steel supports. The abutment zone surrounding the excavation was generally covered with timber lagging or corrugated sheets. Hence, it was difficult to examine whether localized failure (fracturing) has taken place at the abutment. However, on the basis of the observed conditions of the steel arch support, there appears to be no evidence of collapse of the abutment zone.

4.2.1.2. Analysis of Potential Failure at the Roof of the Intersections

The analysis of potential failure at the roof of the intersections was carried out on two vertical sections.

a) a vertical section (6 m wide and 4 m high) perpendicular to the direction of the main roadway (slope). This section was taken in view of the fact that the support for the intersection was installed prior to the excavation of the cross cut. In essence, the intersection support would initially be supporting the span of the main roadway only.

b) a vertical section (8 m wide and 4 m high) passing through the centre of the intersection and pillar corners. This section is derived on the basis of the assumption that when the cross cut has been developed, the effective span at the intersection could be increased to one that is represented by the diagonal dimensions of the intersection. It is recognized that the support will have been installed at this stage.

In each case, the opening was analysed as a two dimensional problem under plane strain conditions. The depth of the opening was assumed to be at a depth of 630 m.

Two types of analysis were carried out to assess the potential overstressed zone above the roof of the intersection for the two cross sections indicated above.

The first analysis was an elastic analysis undertaken using the boundary element program and the empirical rock mass failure criterion suggested by Hoek and Brown

(1980). Homogeneous and linearly elastic medium was assumed in the analysis.

The second analysis was performed using the *FLAC program and the mohr-coulomb failure criterion assuming elasto-plastic behaviour of the material. Different material properties of the rock units including the coal, mudstone, siltstone and sandstone were individually modelled. The program can handle the transfer of excess stresses from overstressed elements to adjacent elements. The interaction will continue until equilibrium condition is reached.

Analysis using the Boundary Element Program

The rock mass strength failure criterion used in the analyses was based on the following expression suggested by Hoek and Brown (1980) for jointed rock mass.

$$\overline{\sigma_1} = \sigma_3 + \sqrt{m\,\sigma_3\sigma_c + s\sigma_c^2}$$

where $\overline{\sigma_1}$ = major principal stress (maximum σ_1) that could be sustained by the rock mass
σ_3 = minor principal stress
σ_c = uniaxial compressive strength of intact rock
m,s = rock mass strength parameters

On the basis of this criterion and a given set of σ_c, m, s and σ_3, the maximum principal stress $\overline{\sigma_1}$ that could be substained by the rock mass can be

*FLAC (Fast Lagrange Analysis of Continua) is developed by ITASCA Inc. for geomechanical analysis.

evaluated. The strength/stress ratio is defined as $\overline{\sigma 1}/\sigma 1$, where $\sigma 1$ is the actual major principal stress applied. When the strength/stress ratio is less than unity, the rock mass is considered to be overstressed.

It may be noted from the above expression that when $\sigma 3$ is zero (that is, the minimum principal stress becomes zero), the strength of the rock mass is equal to $\sqrt{s.\sigma c}$, (s is less than 1), which is generally less than the uniaxial compressive strength of intact rock.

The rock mass strength parameters (m,s) were selected on the basis of the rock mass characteristics which include the intact strength of the rock material, the frequency, orientation and condition of the structural discontinuities, and the groundwater conditions. Using the CSIR* Geomechanics Rock Mass Rating (RMR) developed by Bieniaswki (1973), which takes into account the above factors, the range of rock mass rating and the corresponding m and s parameters for the range of geological conditions at the roof of the main roadway intersections is as follows:

Rock Type	CSIR Rock Mass Rating (RMR)	Hoek and Brown Rock Mass Strength Parameters	
		m	s
1. mudstone	65	3.0	0.05
2. layered (laminated) siltstone	80	5.0	0.1
3. massive siltstone sandstone	90	6.5	0.32

This range of m and s parameters were used in conjunction with the other input parameters presented in Table 4.6 to assess the stability of the roof strata for the intersections.

Results of Analyses

The potential overstressed (shear overstress) zones around the intersection based on the above range of rock mass strength parameters are shown on Table 4.7 for the 6 m and 8 m span and for various rock mass characteristics.

The overstressed zone delineates the area within which induced shear failure of the rock would be initiated as a result of the elastic redistribution of the ground stresses after excavation of the underground opening. Rock supports are therefore required to stabilize this zone.

In practice, the steel set support system was installed at some point behind the face and further rock deformation is allowed to take place before equilibrium condition is reached, as illustrated in Figure 4.1. The final zone of loosened rock supported by the steel sets would therefore be larger than predicted because of these effects. Progressive rock loosening could be stabilized by effective blocking and by installing the support close to the face. The current procedure of excavation and supporting the main roadway (slope) prior to the development of the cross cut would help to restrain the rock movement at the intersection when the cross cut is being driven. That is, if the loosened zone for the 6 m span is adequately supported, further progressive loosening will be substantially minimized when the full geometry of the intersection is developed.

Analysis using the FLAC Program

Similar to the boundary element analysis, two vertical cross sections were examined. The FLAC program is a finite difference code which can model different material properties. The model used for the analysis is shown in

Figure 4.13 for the 8 m span. The stratigraphic sequence modelled is also shown on the same figure. The mohr-coulomb failure criterion was used in the analyses. The shear strength parameters (C' and ϕ') used for various rock types were derived from the rock mass rating and are summarized in Table 4.8.

For the purpose of this analysis, the parameters for coal were assumed to be equal to those of mudstone. Other input parameters such as the mechanical properties of the rock, the in situ stress condition and the boundary conditions for the models are presented in Table 4.8.

The purpose of using the FLAC program is to assess the potential overstressed or (plastic zone) above the intersections by taking into account the heterogeneity of the medium surrounding the intersection and the effects of stress transfer from overstressed elements to adjacent rock mass. These features are not available from simple elastic analysis such as the boundary element method.

Results of Analysis

As discussed earlier in this report, the condition of an underground opening will cause redistribution of the ground stresses existing in the rock. Figure 4.14 presents the stress trajectories in the rock mass surrounding the 6 m span for the case where RMR of the roof strata is about 80.

It may be noted that close to the roof of the opening, the confining stress perpendicular to the roof strata is very low and the strata are essentially subjected to a biaxial stress field (stresses perpendicular and parallel to the longitudinal axis of the opening). However, above the roof the conforming stresses would increase to give a triaxial stress state in the rock mass.

The potential overstressed zones (defined by yielding of the material based on the mohr-coulomb condition) at the roof of the opening for the 6 m and 8 m spans are shown in Figure 4.15 for different rock mass rating of the material. As discussed earlier, the program handles the transfer of excess stresses from overstressed areas to adjacent elements. The potential overstressed zones shown in Figure 4.15 represent the equilibrium condition after stress readjustment.

In general, the extent of the potential failure zone increase with increases in span and reduction in the rock mass rating (reduction in overall rock mass quality) as expected. The shape of the failure zones varies from a triangular shape for higher quality rock mass (RMR=90) to an elliptical shape for lower quality rock mass (RMR=65, 80). The depth of potential overstressed zone for each of the cases is shown in Figure 4.15 and summarized in Table 4.9.

4.2.1.3 Discussions

The potential failure zone at the roof of a typical roadway intersection in the Lingan Mine has been analysed by:

(a) back analysis from the observed (qualitative) deflections of the collar beams and roof beams which are the main structural members for the roadway intersection support systems

(b) stress analysis of the openings pertaining to the intersection geometry

(c) empirical design method

The results obtained from these analyses have been presented in the previous sections. A discussion of the results is presented below.

Table 4.10 presents a summary of the extent of the potential failure zone evaluated using different methods of analysis. For comparison purposes, the depth of the potential failure zone was expressed as equivalent uniform thickness of loosened rock.

The following results were noted from the comparison :

a) The depth of potential overstressed zone determined by the Boundary Element method is generally smaller than that by the FLAC analysis. This is primarily attributed to the difference in the method of analysis.

   In the Boundary Element analysis, homogeneous medium was assumed and the potential overstressed zone was evaluated based on elastic stresses and the Hoek-Brown failure criterion.

   In the FLAC analysis, however, stress transfer would take place once an element has yielded (stressed beyond its peak strength). The excess stress (difference between the sustained stress and the peak strength) would be distributed to adjacent elements. This process would cause stress readjustment in the entire medium and cause further stress transfer. As a result, the potential overstressed zone would expand until equilibrium is reached at which no further stress readjustment is required.

   It is recognized that the actual process of rock yielding and load sharing at the roof of the intersection is rather complex, the numerical simulation described therefore only provides an approximation of the condition observed.

b) The empirical method suggested by Unal (1983) for estimating loose rock height gives results comparable to those obtained by the FLAC numerical modelling techniques. Unal's method was derived on the basis of observed rockfalls in coal mines and correlations with other empirical methods.

c) All the methods which take into account the rock mass characteristics give fairly consistent results.

d) The depth of loosened rock obtained by back analysing the collar beam deflections at a typical four-way intersection is about 0.6 to 0.8 m. Using this as a basis for calibrating the numerical models and the empirical method, the rock mass ratings corresponding to this depth of loosening are summarized below:

Depth of loosened rock (potential overstressed)	Corresponding RMR values		
0.6 to 0.8 m	BEM	80	(6m span)
		80-90	(8m span)
	UNAL	80-90	(6m span)
		90-92	(8m span)
	FLAC	80-90	(6m span)
		90	(8m span)

It may be noted that for a depth of loosened zone ranging from 0.6 to 0.8 m, the RMR values would range from 80-90 depending on the type of analysis and span considered. This is generally consistent with the observed condition of the rock and the corresponding estimated RMR values for the rock at the roof of typical roadway intersections (see section 3.1.2).

As the support system at the intersections was generally installed during development of the main slope and prior to the cross cut excavation, the effective span for the intersection would probably be closer to 6 m.

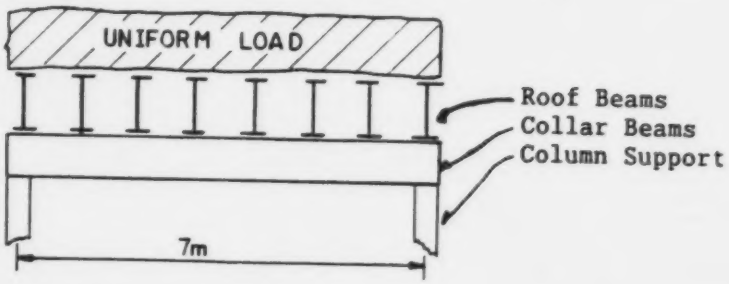
e) The consistency between the results of the analyses, the rock mass rating and the results of back analysis indicates that by using geomechanics classifications, rock mass strength and numerical modelling technique, it would be possible to assess the potential overstressed zone at the roof of the intersection. The depth of overstressed zone using the CBDC empirical design method tends to be conservative for the average condition at the Lingan Mine.

It should be noted that the depth of potential overstressed zone determined by the analyses described above assumes that there is no rock support to the failed material. If rock support is installed as early as possible to provide restraint to the rock, the extent of the potential over-stressed zone could be reduced. The calculated depths of loosened rock indicates that the use of active support system such as rock bolts is applicable to the rock and in situ stress condition for the cases analysed. Further discussion of the application of such support system for the Sydney Coalfield is presented in Section 5.

TABLE 4.1

ESTIMATED ROCK LOAD ON COLLAR BEAM AT A TYPICAL FOUR-WAY INTERSECTION AT THE LINGAN MINE

Beam Deflection at mid span of collar beam d (mm)	Estimated Height of loosened rock over roof beams h (m)	Estimated Total load over intersection (kN)	Estimated Uniformly distributed rock load per beam (kN)	Estimated Maximum Bendings Stresses In Collar Beam (MPa)
30	0.3	340	85	125
60	0.6	680	170	255
80	0.8	907	227	300
100	1.0	1135	284	420
150	1.5	1700	425	600



Assumed loading configuration for rock load calculation.

NOTE:

1. The loaded area used for calculating the height of loosened rock is 7 m x 6 m.
2. Density of rock for rock load calculation is 2.7 Mg/m^3.
3. Four collar beams are used for each four-way intersection.

TABLE 4.2

SUMMARY OF DEFLECTIONS AND MAXIMUM BENDING STRESSES IN ROOF BEAMS AT A FOUR-WAY INTERSECTION

Load Case	Total Load per Beam (kN)	Equivalent Height of Uniformly Distributed Rock Load	Total load over Intersection (kN)	Beam Deflection at mid-span of Roof Beam (mm)	Estimated Maximum Bending Stress in Roof Beam (Mpa)
Uniformly Distributed Load	34	0.3	340	9	50
	68	0.6	680	18	100
	113	1.0	1135	30	166
	170	1.5	1700	45	250
Point Load at mid span	34	0.3	340	14	100
	68	0.6	680	29	200
	113	1.0	1135	48	332
	170	1.5	1700	72	498
Point Loads at Mid and Quarter Span	34	0.3	340	11	66
	68	0.6	680	23	133
	113	1.0	1135	38	222
	170	1.5	1700	57	332
Triangular Distributed Load	34	0.3	340	11	66
	68	0.6	680	22	132
	113	1.0	1135	36	221
	170	1.5	1700	55	332

Notes:
1. The span of the roof beam is 6 m.
2. Density of rock for load calculation = 2700 kg/m^3.
3. See Figure 4.2 for sketches of the loading configurations.

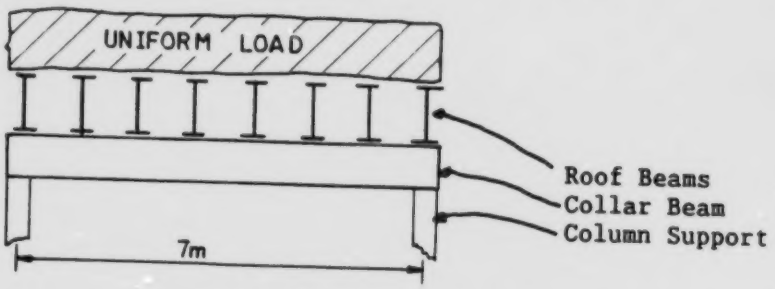
June 1988

871-1369

TABLE 4.3

ESTIMATED ROCK LOAD ON COLLAR BEAM AT A TYPICAL THREE-WAY INTERSECTION AT THE LINGAN MINE

Beam Deflection at mid span of collar beam d (mm)	Estimated Height of loosened rock over roof beams h (m)	Total load over intersection (kN)	Estimated Uniformly distributed rock load per beam (kN)	Estimated Maximum Bendings Stresses in Collar Beam (MPa)
30	0.3	243	122	30
60	0.6	486	243	60
110	1.0	810	405	100
160	1.5	1215	608	150



Assumed loading configuration for rock load calculation.

NOTE: 1. The loaded area used for calculating the height of loosened rock is 6 m x 5 m.

2. Density of rock for rock load calculation is 2700 Kg/m^3.

June 1988

871-1369

TABLE 4.4

SUMMARY OF DEFLECTIONS AND MAXIMUM BENDING STRESSES IN ROOF BEAMS AT A THREE-WAY INTERSECTION

Load Case	Total Load on Beam (Kn)	Equivalent Height of Uniformly Distributed Rock Load	Total load over Intersection (Kn)	Beam Deflection at mid-span of roof beam (mm)	Estimated Maximum Bending in Roof Beam (Mpa)
Uniformly Distributed Load	22	0.3	243	3	27
	44	0.6	486	7	54
	74	1.0	810	11	90
	110	1.5	1215	17	135
Point Load at mid span	22	0.3	243	5	54
	44	0.6	486	11	108
	74	1.0	810	18	180
	110	1.5	1215	26	270
Point Loads at Mid and Quarter Span	22	0.3	243	4	36
	44	0.6	486	8	72
	74	1.0	810	14	120
	110	1.5	1215	21	180
Triangular Distributed Load	22	0.3	243	4	36
	44	0.6	486	8	72
	74	1.0	810	14	120
	110	1.5	1215	21	180

Notes:
1. The span of the roof beam is 5 m.
2. Density of rock for load calculation = 2700 Kg/m^3.
3. See Figure 4.2 for sketches for the loading configurations.

June 1988

871-1369

TABLE 4.5

SUMMARY OF INPUT PARAMETERS FOR NFOLD MODELLING

1. **Model Dimensions**

Width of element	2 m	(Along strike)
Height of element	2 m	(Vertically projected)
Thickness of element	3.7 m	
Dip of Strata	7°	

2. **In Situ Stresses**

σ_v = 0.027 z (MPa)

σ_h W-E = $2x\sigma_v$ = 0.054 z (MPa) West-East Direction (Strike Direction)

σ_hN-S = 0.027 z (MPa) North-South direction (Perpendicular to strike direction)

where z = depth in meters

3. **Rock and Seam Properties**

Deformation Properties

E (rock) = E siltstone = 36 GPa (ref. Table 3.1)

*E (seam) = E equivalent = 5.6 GPa

ν(rock) = ν(seam) = 0.20

*NOTE: The NFold model was used for elastic analysis.

May 1988

871-1369

TABLE 4.6

SUMMARY OF INPUT PARAMETERS FOR BOUNDARY ELEMENT MODELLING

1. Model Dimension

   Drift dimension: 6 m width and 8 m width
   4 m height

2. In situ Stresses*

   σ_V = 17 MPa

   σ_H = 34 MPa

3. Rock Properties

   E (rock) = 36 GPa

   ν = 0.20

   Compressive strength = 48 MPa

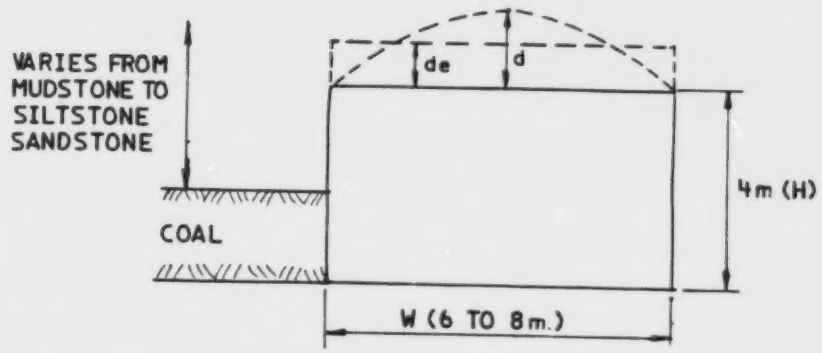
*Analysis in homogeneous media

June 1988

871-1369

Table 4.7

DEPTH OF POTENTIAL OVERSTRESSED ZONE
AT THE ROOF OF ROADWAY INTERSECTION
IN THE LINGAN MINE
(Based on Elastic Boundary Element Analysis)



Case	W	H	Rock Type at Roof	RMR*	Strength Parameters m	s	Depth of Overstressed Zone d	de
1	6	4	mudstone	65	3.0	0.05	2.4	1.2
	6	4	interbedded siltstone	80	5.0	0.1	1.5	0.75
	6	4	massive sandstone siltstone	90	6.5	0.32	0.9	0.45
2	8	4	mudstone	65	3.0	0.05	>3.0	>1.5
	8	4	interbedded siltstone	80	5.0	0.1	2.4	1.2
	8	4	massive sandstone siltstone	90	6.5	0.32	1.2	0.6

1. *RMR of rock above roof of intersection.

2. de = equivalent approximate uniform thickness of loosened rock.

June 1988

871-1369

TABLE 4.8

MECHANICAL PROPERTIES USED IN FINITE DIFFERENCE MODELLING

	Sandstone	Siltstone	Mudstone	Coal
E (GPa)	30.34	36.55	15.44	15.44
ν	0.18	0.20	0.11	0.11
k (GPa)	15.80	20.31	6.60	6.60
G (GPa)	12.86	15.23	6.96	6.96
ρ (kg/m^3)	2700	2700	2700	1350
σ_c (MPa)	77.91	48.23	20.69	11.79
RMR = 90 m = 7 s = 0.3				
c'(MPa)	8.8°	5.6	2.8	2.8
ϕ'	48°	45°	40°	40°
RMR = 80 m = 5 s = 0.1				
c (MPa)	5.0	3.1	1.5	1.5
ϕ'	48°	45°	40°	40°
RMR = 65 m = 3 s = 0.02				
c (MPa)	2.2	1.4	0.75	0.75
ϕ	45°	40°	35°	35°

In situ stresses

σv = 17 MPa
σh = 34 MPa

Legend

E = Elastic modulus
k = bulk modulus
G = shear modulus
ρ = density
σc = uniaxial compressive strength
c' = effective cohesion
ϕ' = effective angle of internal friction

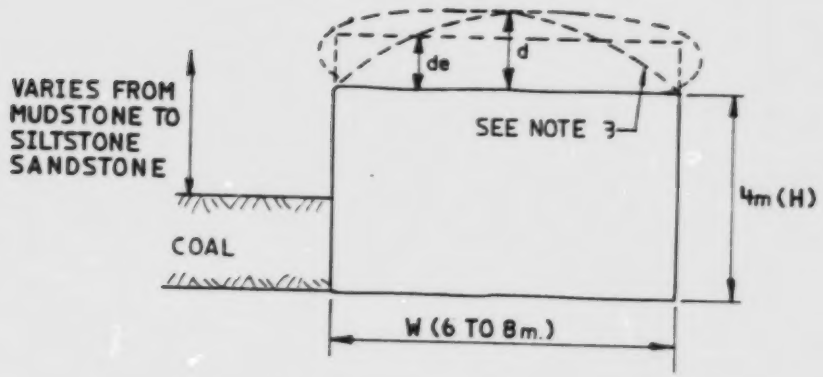
June 1988

871-1369

TABLE 4.9

DEPTH OF POTENTIAL OVERSTRESSED ZONE AT THE ROOF OF ROADWAY INTERSECTION IN THE LINGAN MINE

(Based on FLAC Analysis)



Case	W	H	Rock Type at roof	RMR	Strength Parameters C' (MPa)	φ'	d	de
1	6	4	mudstone	65	1.4	40°	3.5	3.5
	6	4	interbedded siltstone	80	3.1	45°	2	1.0
	6	4	massive sandstone siltstone	90	5.6	45°	1.4	0.7
2	8	4	mudstone	65	1.4	40°	4.5	4.0
	8	4	interbedded siltstone	80	3.1	45°	2.8	1.8
	8	4	massive sandstone siltstone	90	5.6	45°	1.8	1.0

NOTES:

1. d = maximum height of potential overstressed zone
2. de = equivalent uniform thickness of potential overstressed zone
3. Refer to Figure 4.15 for approximate shape of potential overstressed zone

June 1988

871-1369

Table 4.10

ESTIMATE OF POTENTIAL FAILURE ZONE ABOVE MAIN ROADWAY INTERSECTION

Method of Analyses			Estimated Depth of Potential Overstressed Zone (m) d	de
1. Back Analysis	Based on collar beam deflection due to loading over a 7m x 6m area			0.6-0.8m
2. Boundary Element Analysis	6m span	(RMR=65)	2.4	1.2
		(RMR=80)	1.5	0.75
		(RMR=90)	0.9	0.45
	8m span	(RMR=65)	3.5	1.8
		(RMR=80)	2.4	1.2
		(RMR=90)	1.2	0.6
3. FLAC Analyses	6m span	(RMR=65)	3.5	3.5
		(RMR=80)	2.0	1.0
		(RMR=90)	1.4	0.7
	8m span	(RMR=65)	4.5	4.0
		(RMR=80)	2.8	1.8
		(RMR=90)	1.8	1.0
UNAL's* Empirical Design Method	6m span	(RMR=65)	-	2.1
		(RMR=80)	-	1.2
		(RMR=90)	-	0.6
	8m span	(RMR=65)	-	2.8
		(RMR=80)	-	1.6
		(RMR=90)	-	0.8
4. CBDC empirical Design	6m span		3	1.5
	8m span		4	2

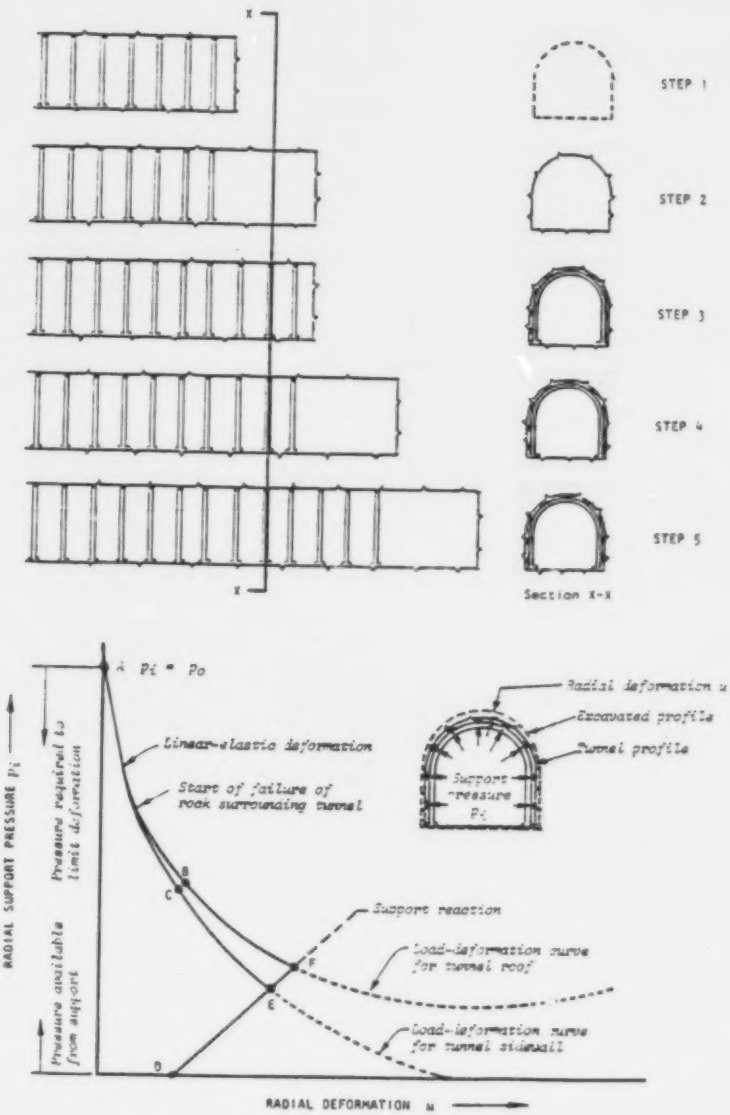
Notes: d = maximum depth of potential overstressed zone

de = equivalent depth of uniform load

* = refer to Section 2 for Unal's empirical method of estimating potential overstressed zone

HYPOTHETICAL LOAD - DEFORMATION CURVES FOR THE ROCK MASS AND THE SUPPORT

FIGURE 4.1



REF: HOEK AND BROWN (1980)

Date APRIL 12, 1988

Project 871-1369

Golder Associates

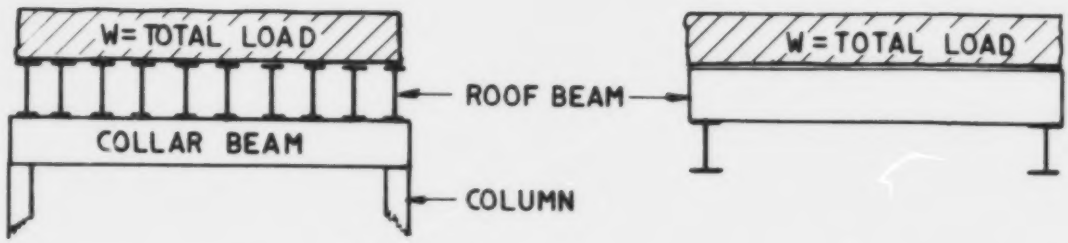
Drawn D. M.

Chkd F.M.

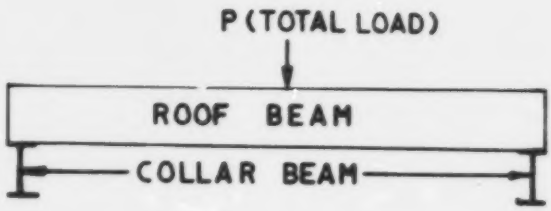
Form G.A.-D-4m (metric)

TYPICAL LOADING CONFIGURATIONS FOR COLLAR AND ROOF BEAMS

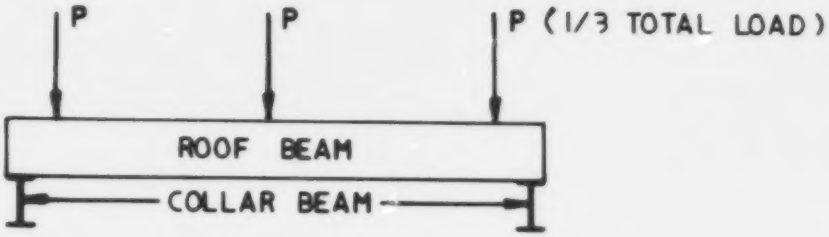
FIGURE 4.2



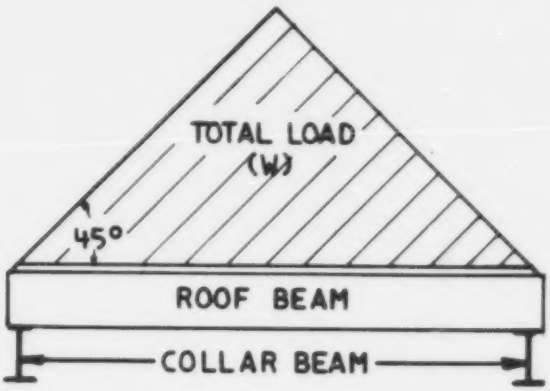
a) UNIFORMLY DISTRIBUTED LOAD



b) POINT LOAD AT MID SPAN



c) POINT LOADS AT MID AND QUARTER POINT



d) TRIANGULAR DISTRIBUTED LOAD

Date MAY 31, 1988
Project 871-1369

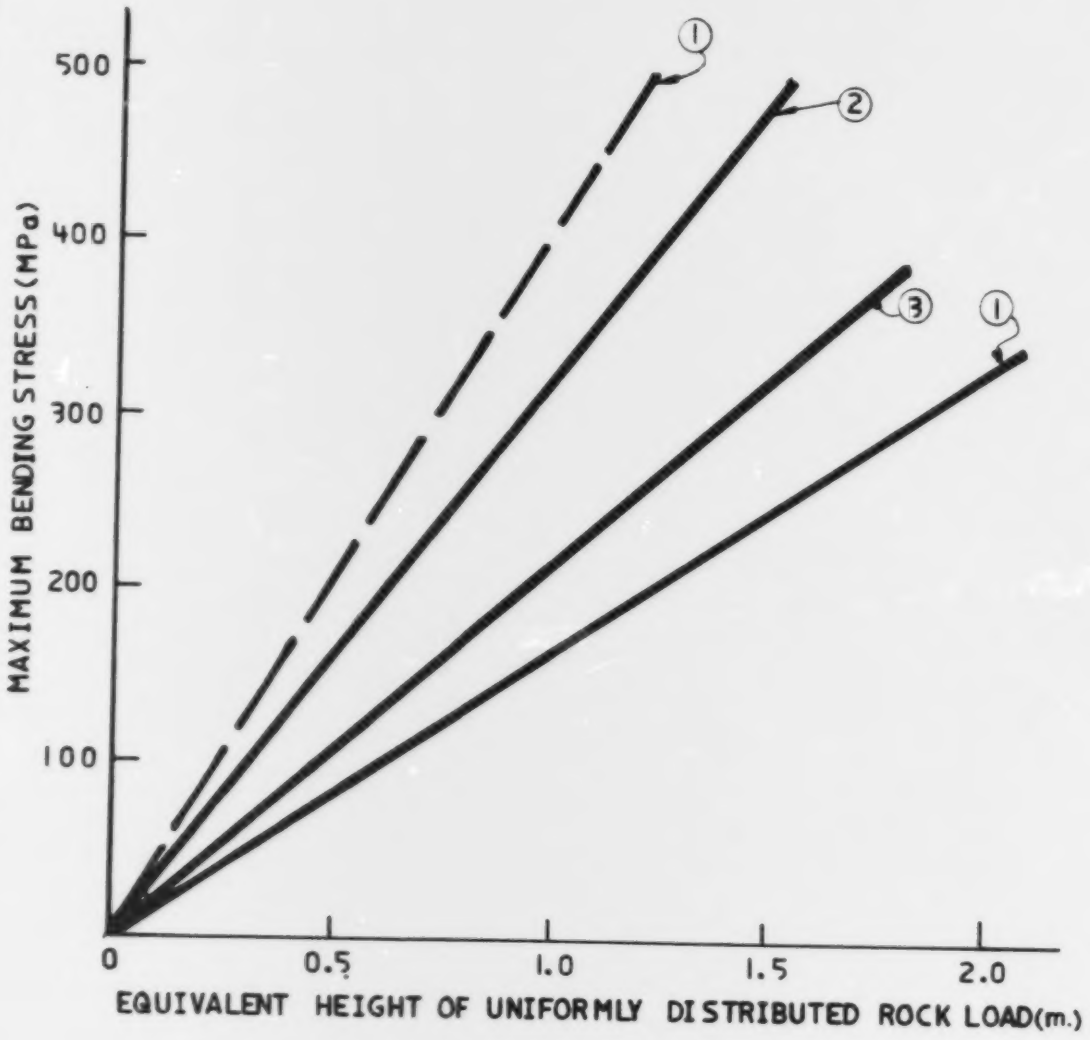
Golder Associates

Drawn D. M.
Chkd F.M.

Form GA-D-4m (metric)

PLOT OF THE DIFFERENT BENDING STRESSES CAUSED BY EQUIVALENT UNIFORM LOAD FOR THE ROOF AND COLLAR BEAM (FOUR-WAY INTERSECTION)

FIGURE 4.3



LEGEND

- COLLAR BEAM (W200X59) 7m
- ROOF BEAM (W200X52) 6 m
- ① UNIFORM LOADING
- ② POINT LOADING
- ③ TRIANGULAR LOADING

Date APRIL 25, 1988
Project 871-1369

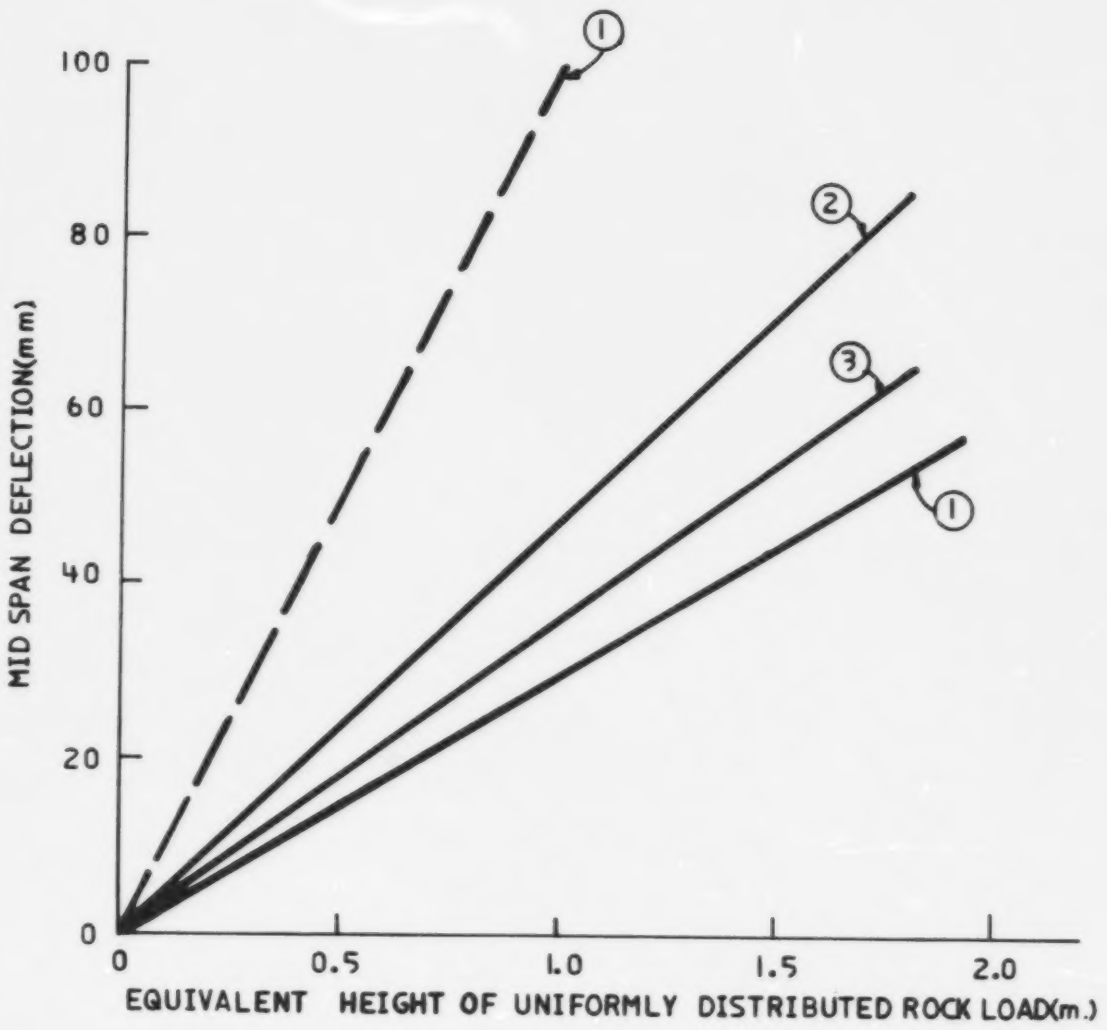
Golder Associates

Drawn D. M.
Chkd F.M

Form GA-D-4m (metric)

PLOT OF THE DIFFERENT MID SPAN DEFLECTIONS CAUSED BY EQUIVALENT UNIFORM LOAD FOR THE ROOF AND COLLAR BEAM
(FOUR-WAY INTERSECTION)

FIGURE 4.4



LEGEND

COLLAR BEAM (W200X59)(7m.)

ROOF BEAM (W200X52)(5 m.)

① UNIFORM LOADING

② POINT LOADING AT MID SPAN

③ TRIANGULAR LOADING

Date APRIL 21, 1988

Project 871-1369

Golder Associates

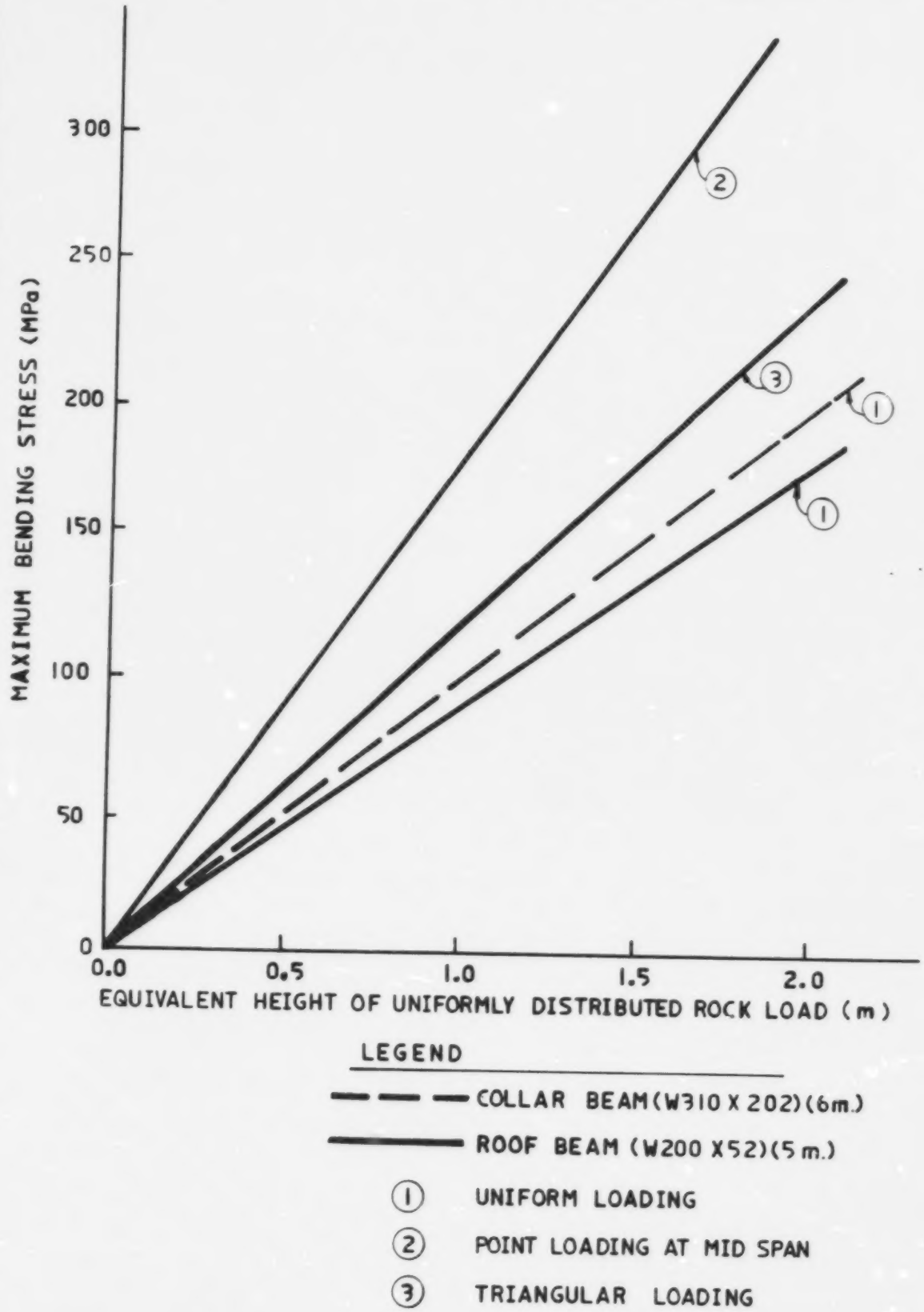
Drawn D.M.

Chkd

Form G.A.-D-4m (metric)

PLOT OF DIFFERENT BENDING STRESSES CAUSED BY EQUIVALENT UNIFORM LOAD FOR ROOF AND COLLAR BEAM (THREE-WAY INTERSECTION)

FIGURE 4.5



LEGEND

COLLAR BEAM (W310 X 202) (6m.)

ROOF BEAM (W200 X 52) (5 m.)

(1) UNIFORM LOADING

(2) POINT LOADING AT MID SPAN

(3) TRIANGULAR LOADING

Date APRIL 25, 1988

Project 871-1369

Golder Associates

Drawn D. M.

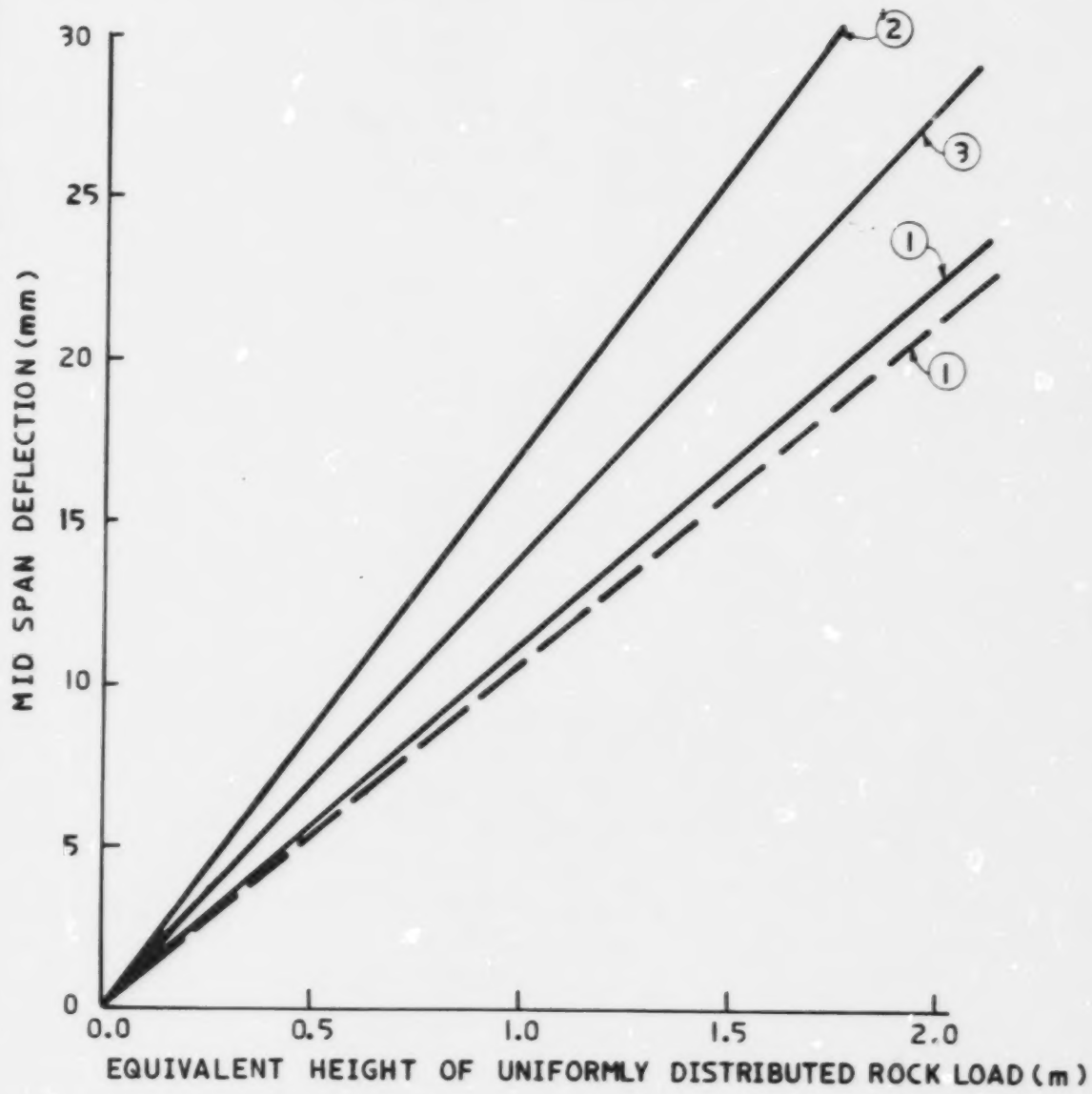
Chkd F.M.

FORM PROVIDED JUNE 1989

Form G.A.-D-4m (metric)

PLOT OF THE DIFFERENT MID SPAN DEFLECTIONS CAUSED BY EQUIVALENT UNIFORM LOAD FOR ROOF AND COLLAR BEAM (THREE-WAY INTERSECTION)

FIGURE 4.6



LEGEND

COLLAR BEAM (W310X202) (7m.)

ROOF BEAM (W200X52) (6m.)

① UNIFORM LOADING

② POINT LOADING AT MID SPAN

③ TRIANGULAR LOADING

Date APRIL 25, 1988

Project 871-1369

Golder Associates

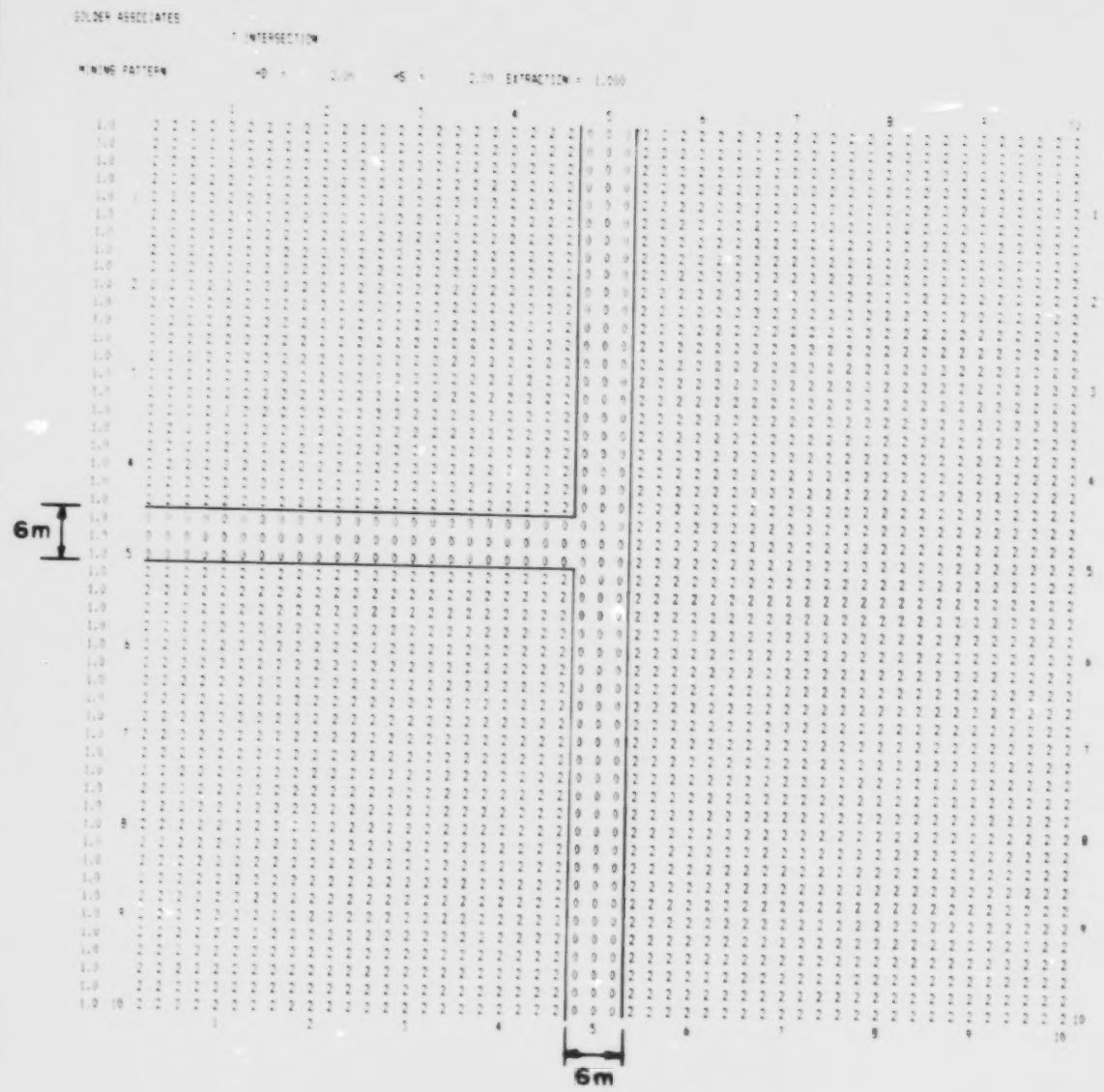
Drawn D.M.

Chkd F.M.

Form GA D-4m (metric)

MINING PATTERN - LINGAN MINE
TYPICAL 3 WAY "T" INTERSECTION

FIGURE 4.7



LEGEND

DRIFT AREA

ELEMENT 2x2m

Date MAY 26, 1988

Project 871-1369

Golder Associates

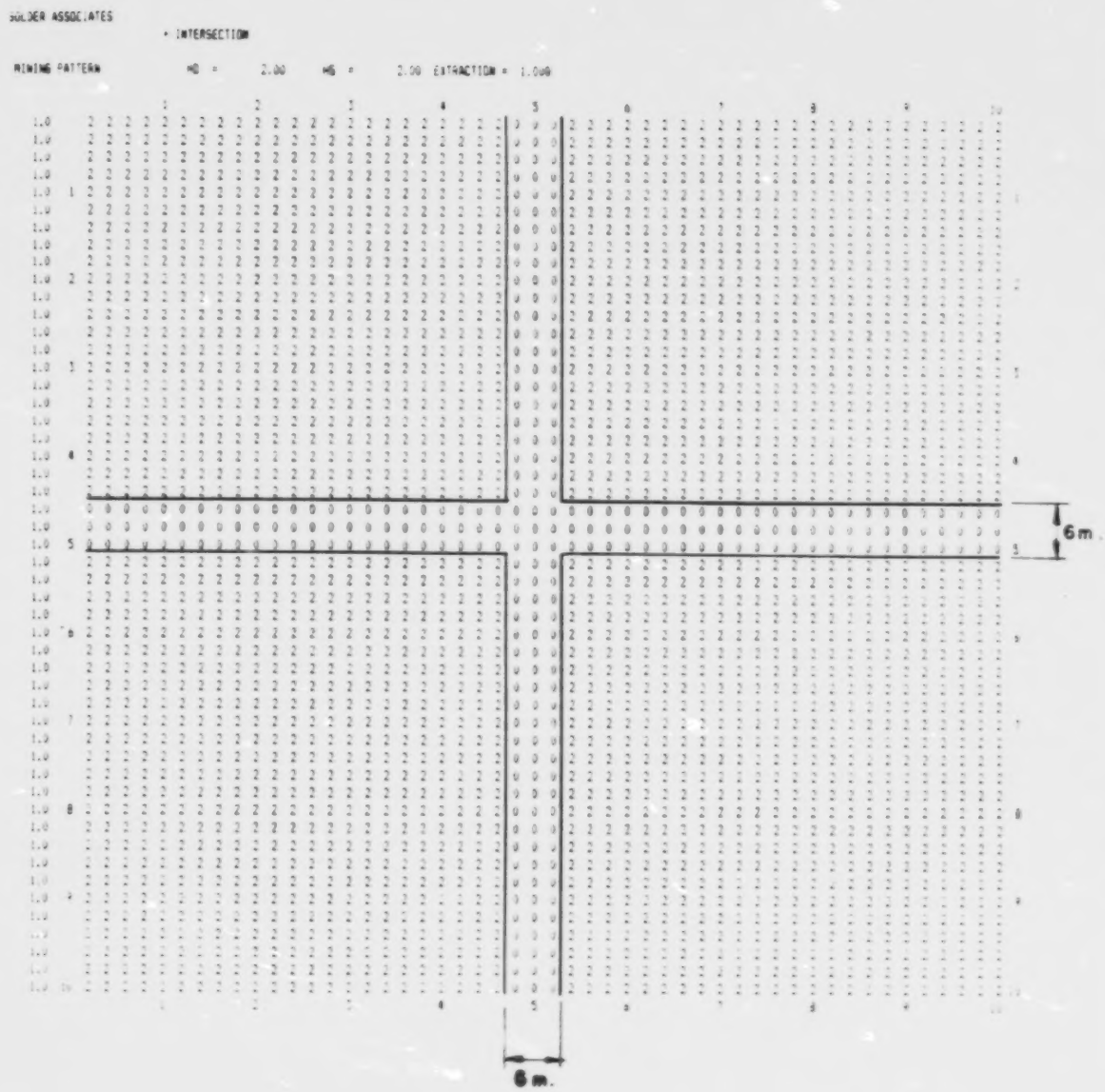
Drawn D.M.

Chkd F.M.

Form GA-D-4m (metric)

MINING PATTERN - LINGAN MINE
TYPICAL 4-WAY INTERSECTION

FIGURE 4.8



LEGEND

DRIFT AREA

ELEMENT 2 x 2 m

Date MAY 26, 1988
Project 871-1389

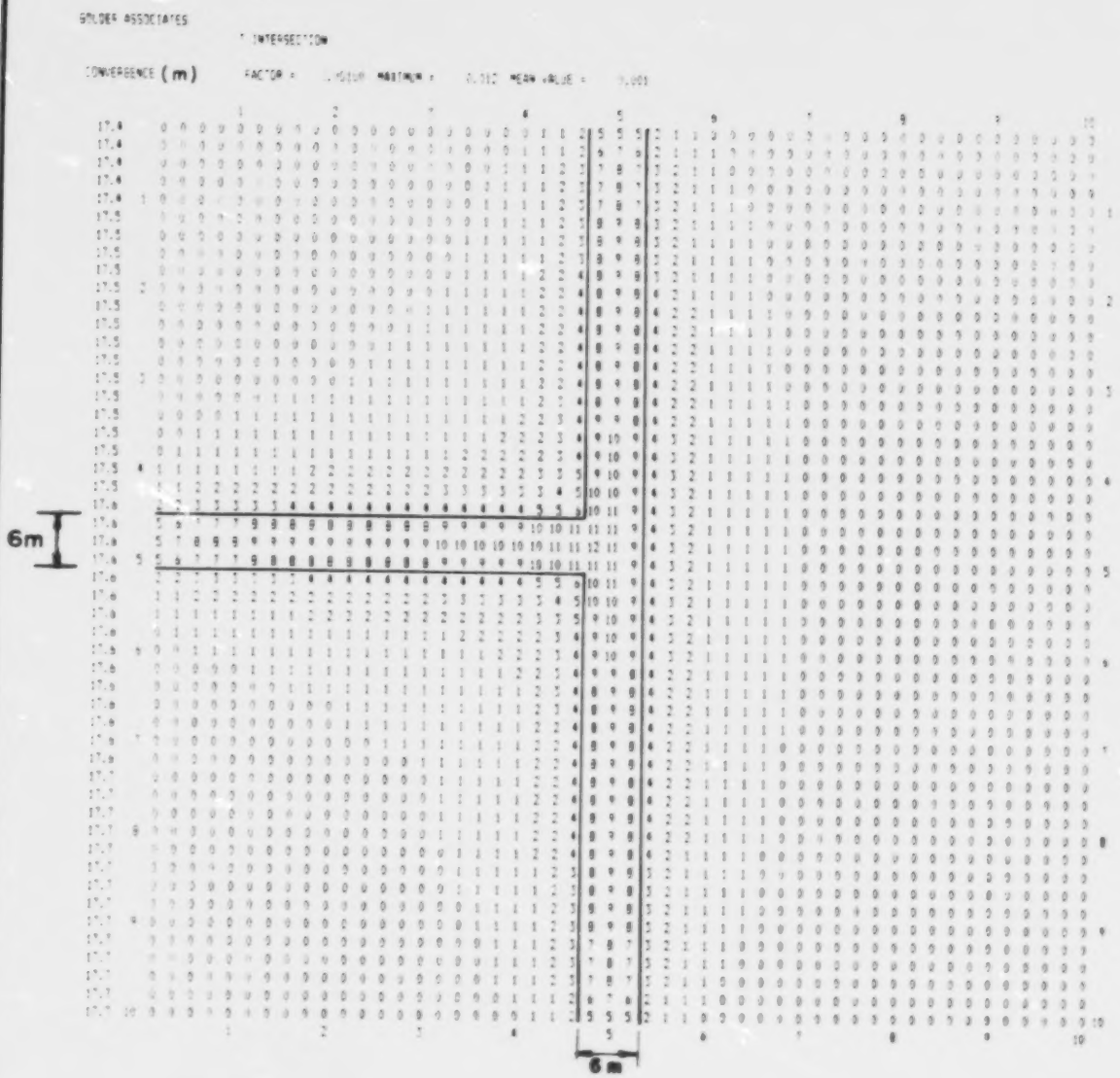
Golder Associates

Drawn D.M.
Chkd F.M.

Form GA-U-4m (metric)

CONVERGENCE PATTERN-LINGAN MINE
TYPICAL 3-WAY "T" INTERSECTION

FIGURE 4.9



LEGEND

DRIFT AREA

ELEMENT 2x2 m.

Date MAY 26, 1988
Project 871-1369

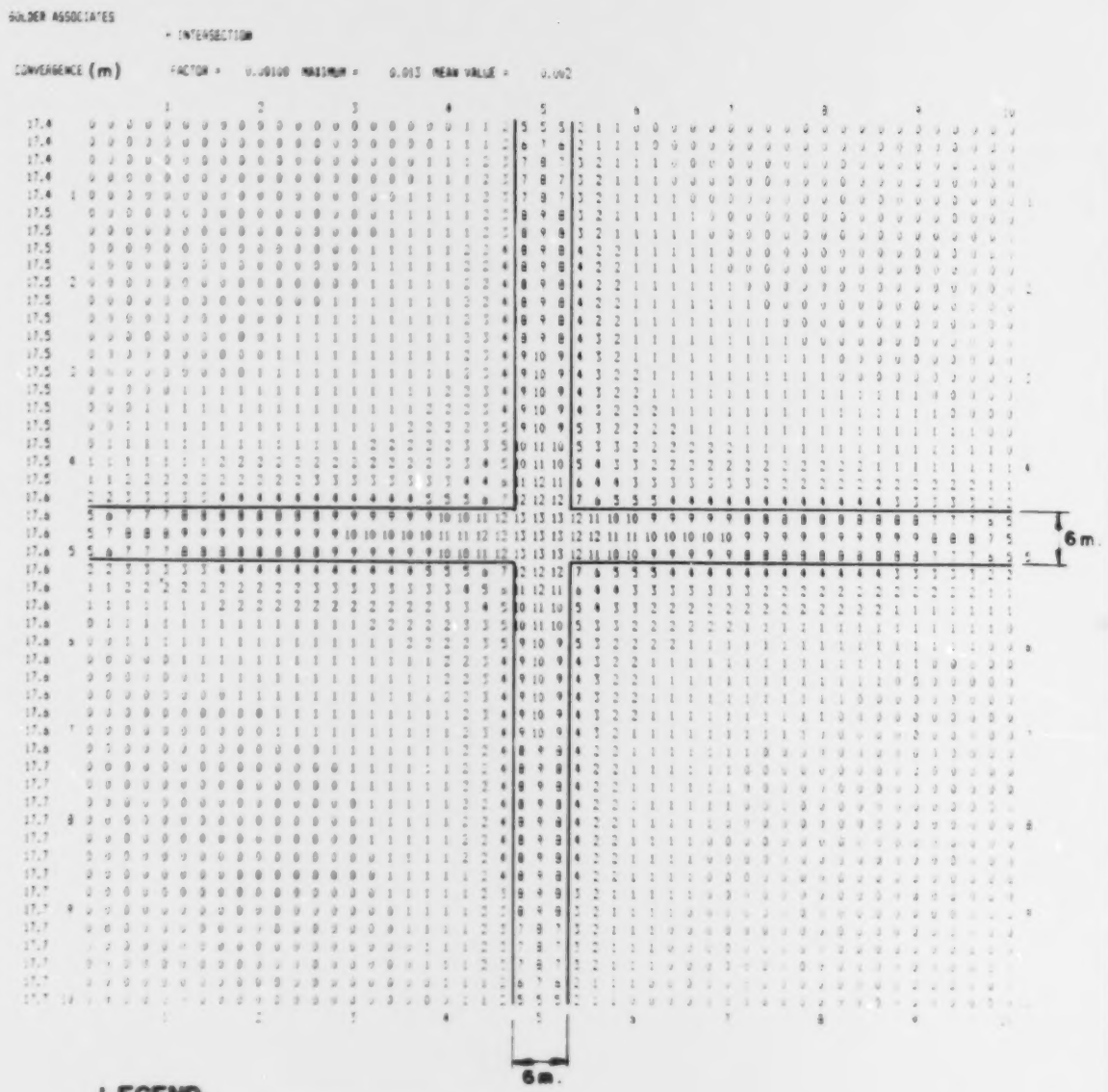
Golder Associates

Drawn D. M.
Chkd F.M.

Form GA-D-4m (metric)

CONVERGENCE PATTERN - LINGAN MINE
TYPICAL 4-WAY INTERSECTION

FIGURE 4.10



LEGEND

DRIFT AREA

ELEMENT 2x2m

Date MAY 26, 1988

Project 871-1389

Golder Associates

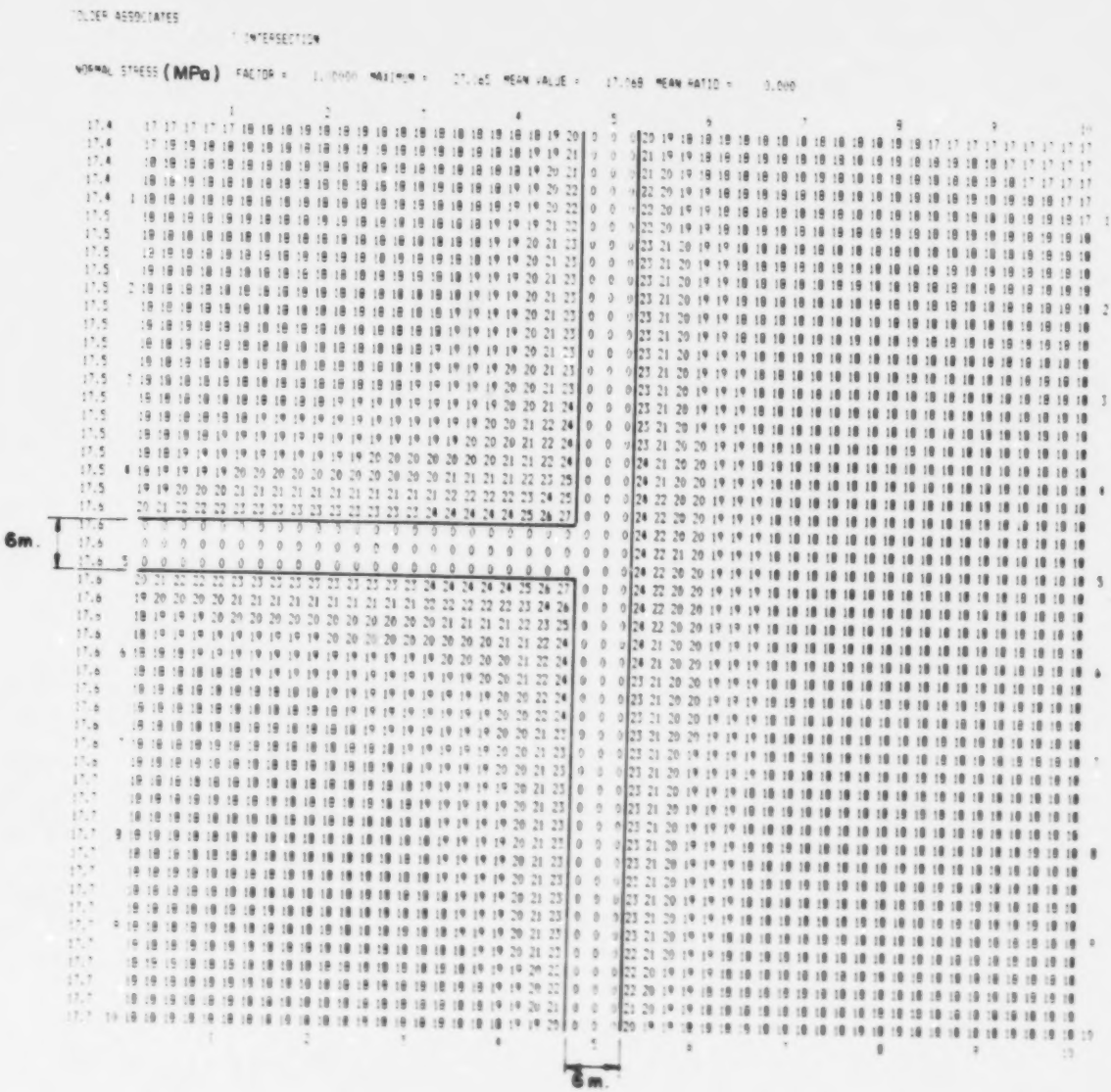
Drawn D.M.

Chkd F.M.

Form GA-D-4m (metric)

NORMAL STRESS PATTERN
TYPICAL 3 WAY "T" INTERSECTION

FIGURE 4.11



LEGEND

DRIFT AREA

ELEMENT 2x2m

Date MAY 26, 1988

Project 871-1369

Golder Associates

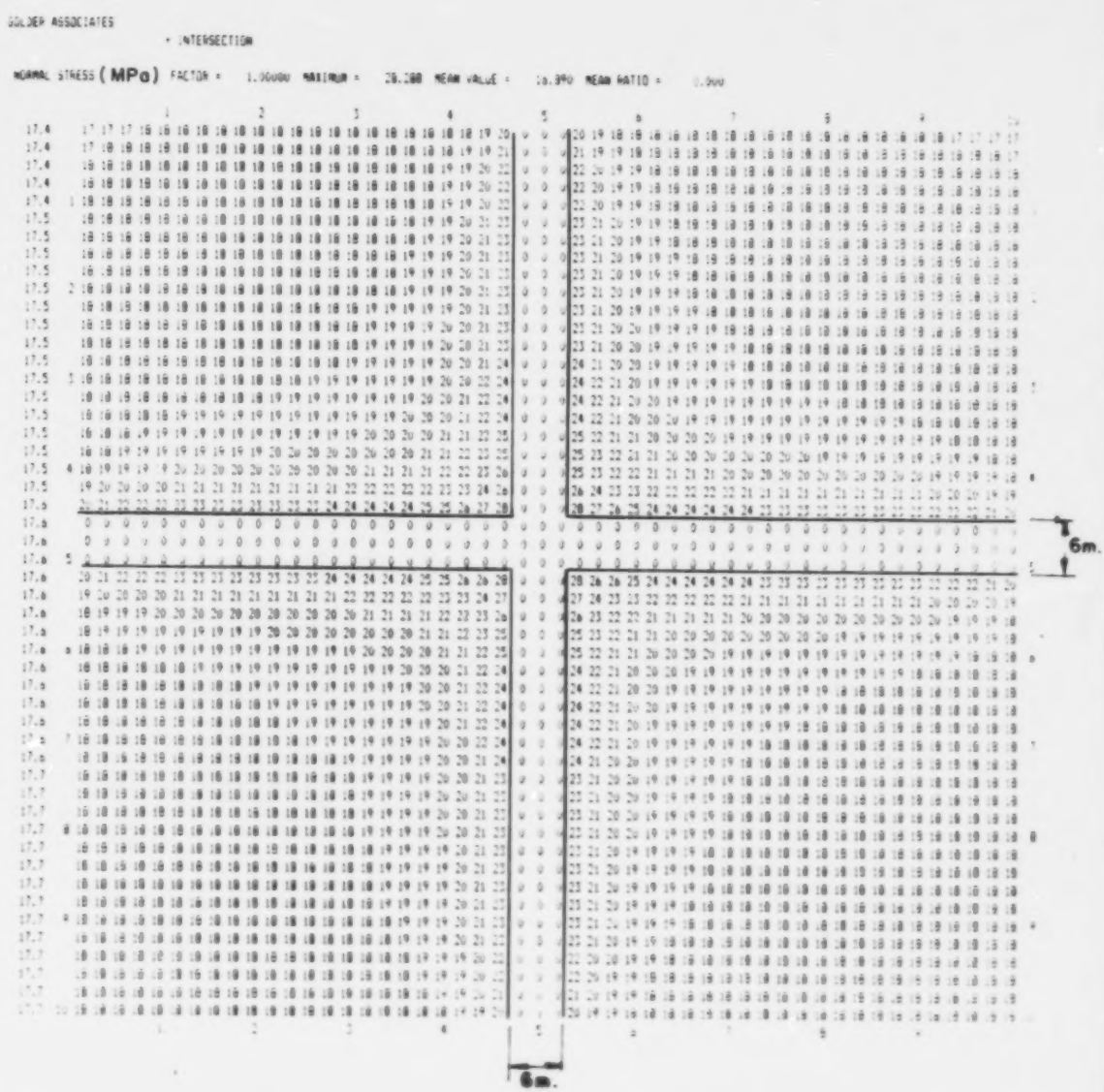
Drawn D.M.

Chkd F.M.

Form GA-D-4m (metric)

NORMAL STRESS PATTERN
TYPICAL 4 WAY INTERSECTION

FIGURE 4.12



LEGEND

DRIFT AREA

ELEMENT 2x2m

Date MAY 26, 1988

Project 871-1389

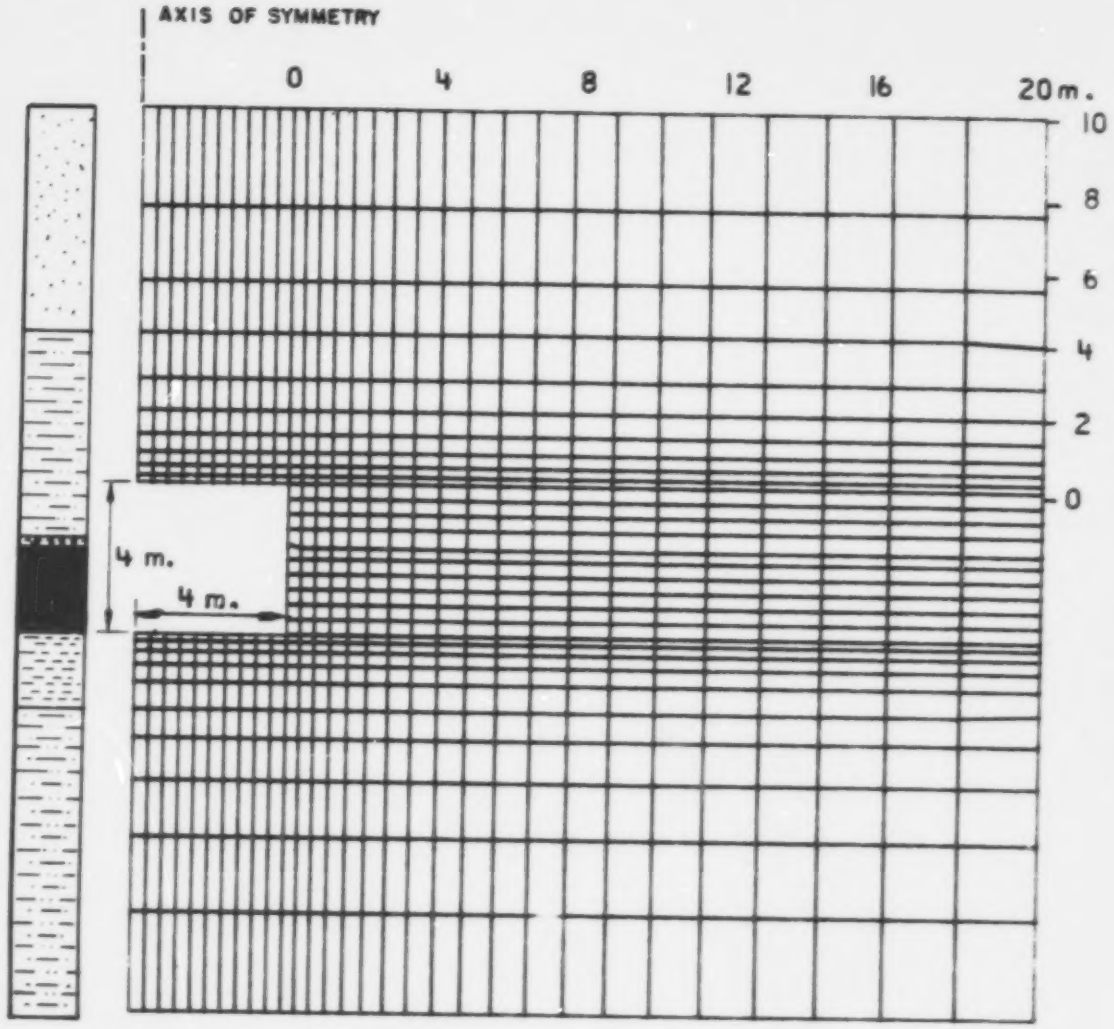
Golder Associates

Drawn D. M.

Chkd P.A.M.

FINITE DIFFERENCE GRID SHOWING TYPICAL GEOLOGY (LINGAN MINE)

FIGURE 4.13



LEGEND

- SANDSTONE
- SILTSTONE
- MUDSTONE
- COAL

0 1 2 3 4 5
SCALE, m.

Date APRIL 20, 1988
Project 871-1369

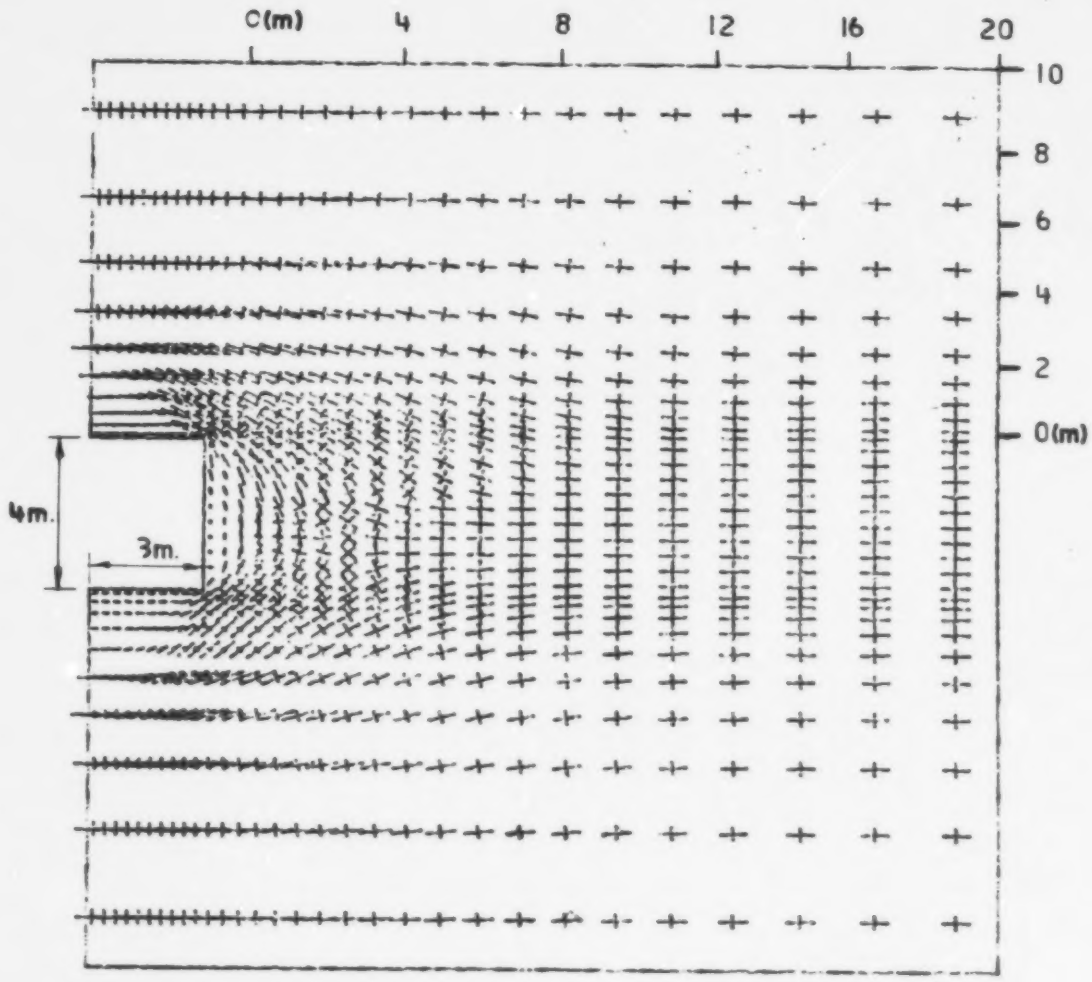
Golder Associates

Drawn D.M.
Chkd

Form G.A.-D-4m (metric)

STRESS TRAJECTORIES AROUND AN INTERSECTION
(BASED ON FLAC ANALYSIS)

FIGURE 4.14



LEGEND

SCALE

0 1 2 3 4 5(m)

PRINCIPAL STRESSES
MAXIMUM STRESS - 70 MPa

0 200 (MPa)

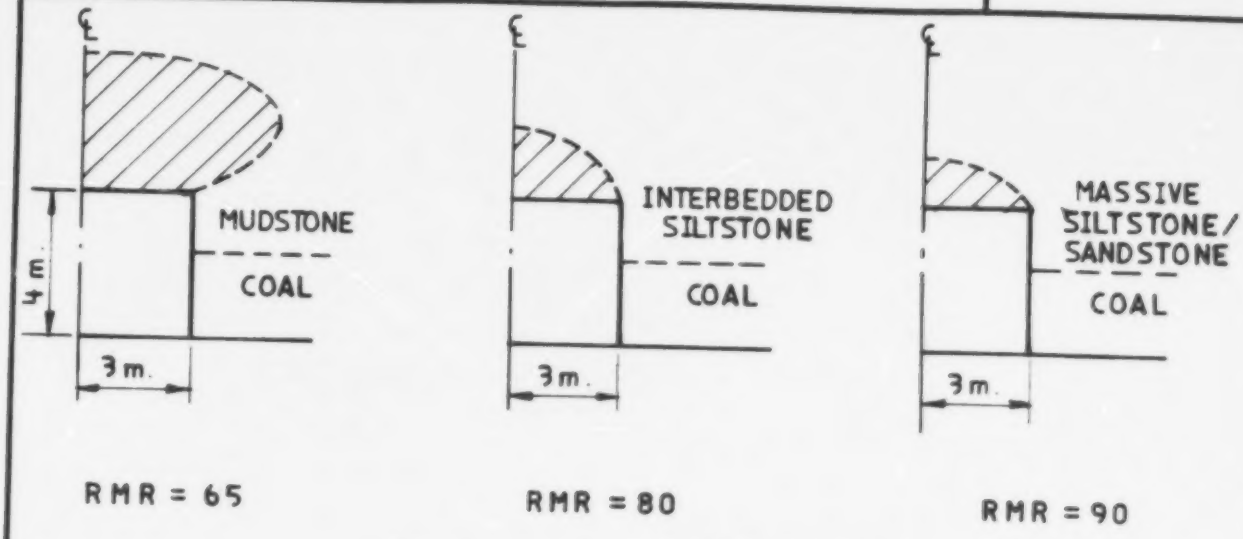
Date MAY 30, 1988
Project 871-1369

Golder Associates

Drawn D. M.
Chkd

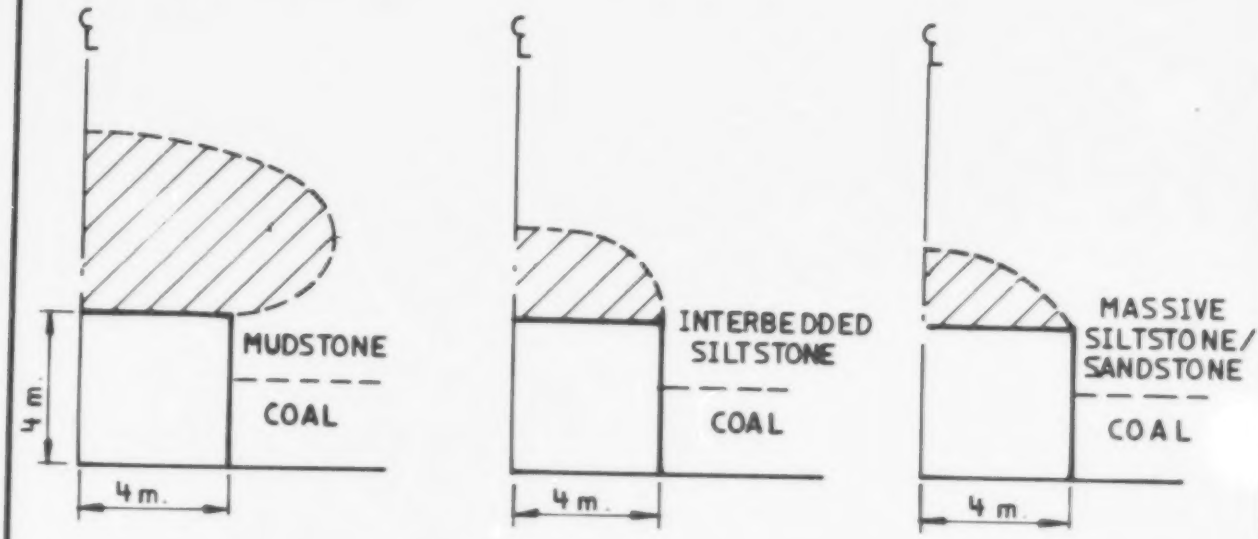
POTENTIAL OF OVERSTRESSED ZONE AT THE ROOF OF INTERSECTION
(BASED ON RESULTS OF FLAC ANALYSES)

FIGURE 4.15



RMR = 65 RMR = 80 RMR = 90

CASE 1 - 6 m. SPAN OPENING



RMR = 65 RMR = 80 RMR = 90

CASE 2 - 8 m. SPAN OPENING

LEGEND

POTENTIAL OVERSTRESSED ZONE

NOTE

THE OVERSTRESSED ZONE WAS DETERMINED BY USING FLAC WITH MOHR-COULOMB FAILURE CRITERION FOR THE MATERIAL.

SCALE 0 5 (m)

Date MAY 30, 1988
Project 871-1369

Golder Associates

Drawn DM
Chkd FM

5. APPLICATION OF ROADWAY INTERSECTION DESIGN TECHNIQUES TO THE SYDNEY COALFIELD

The roadway intersection design techniques used in different countries have been reviewed. In general, the rock conditions of coal measures strata in other coal mining areas are not significantly different from those encountered in the Sydney Coalfield in terms of its broad lithological characteristics. The shale and sandstone strata in the US coal mines are comparable in strength to the siltstone and sandstone units in the Sydney coalfield. The in situ stress conditions in the coal mines reviewed indicate a slightly elevated horizontal stress field with some anistrophy in the stresses on the horizontal plane. Similar stress conditions exist in the Sydney coalfield.

Despite the similarities between these coal mining areas in broad terms, site specific conditions often control the stability of the roadways and the design of the support system. In general, the design philosophy for roadway intersection support falls into three categories:

(a) use of active support system such as rock bolts designed according to the rock conditions, span and type of rock bolts used (eg. Peng et al 1978)

(b) use of passive support system such as steel sets designed according to precedent practice and performance experience

(c) A combination of rock bolts and steel sets on timber posts (eg. Unal, 1983)

Each of these categories of supports have been successfully practised in roadways in a range of coal mining environment. Some of these experiences may not be directly transferable to the Sydney coalfield because of site specific conditions but the experience gained and approaches used by others are useful references for the consideration of viable alternative support systems by the Cape Breton Development Corporation. In addition, rock support guidelines used by other countries are also useful for design considerations.

A significant point emerged from the comparison of the observed loosened rock height and the potentially overstressed zone evaluated by the analytical and empirical methods. Consistent results are obtained by taking into account the rock mass characteristics of the strata in which the opening is to be developed. This is one of the key factors which determine the support requirement for the opening.

The analyses presented in Section 4 also indicate that the height of loosened rock at a typical intersection with siltstone roof at the Lingan Mine (RMR at the roof = 80-90) is such that roof bolting would be a viable alternative support to heavy steel sections.

The increasing tendency to use heavier support in the operating mines in the Sydney Coalfield in response to the observed performance of the support (eg. beam deflections) should be re-examined by considering (a) the need to justify the support used, (b) other factors which might have influenced the support performance (such as installation procedures and practices) but may not be directly related to rock loading, and (c) the use of an alternative support system.

Consideration should also be given to the procedures for the demonstration and implementation of possible alternative support systems. Recognizing the geological environment and the operating conditions in the Sydney Coalfield, a phased approach would be most appropriate for enhancing the current support design methodology and practices. The suggested approach would comprise four phases as outlined below.

(a) Phase 1
characterization of selected existing intersections

(b) Phase 2
performance monitoring of a new intersection supported by current support systems

(c) Phase 3
Implementation and monitoring of an experimental program to demonstrate the performance of support alternatives

(d) Phase 4
Implementation and monitoring of a full scale intersection using the enhanced support system derived from phases 2 and 3

Phase 1

This phase entails a detailed characterization of several selected main roadway intersections representative of the range of rock conditions in future mining areas. The characterization will include the following:

- Determination of the height of loosened rock above the intersection support. This can be accomplished by visual examination and augmented by probe drilling if required. The purpose is to ascertain the profile

of rock loosening in relation to the geological condition and the support used.

- Detailed examination of the condition of the support system including the blocking details, the deflection of the roof and collar beams (relative to the supporting columns or beams),

- Back calculation of the loading on the roof and collar beams using the procedures described in Section 4. If the deflections are known at different points on the beams and the end conditions are better defined, a more refined structural analysis could be performed.

- Comparison of the back calculated loading with the zone of loosened rock observed and with the assumed rock load for the design of the roadway, the results from this phase would provide a reference condition for the consideration of enhancing the current support practice and implementing an alternative support system.

Phase 2

This phase consists of instrumenting a new intersection to be supported by the current support system. The work entailed would comprise the following:

- Define the stratigraphy at the intersection based on available geological information.

- Collect data on mechanical properties of the rock strata encountered at the intersection.

- Conduct geological mapping of the exposed rock condition during excavation of the intersection. A

detailed record of the face and roof conditions and the support installation details should be compiled.

- Install extensometers close to the face, at the centre and edges of the span of the roadway. The majority of the anchors should be located within the anticipated zone of loosening

  Extensometers similar to that installed at the Phalen Mine by CANMET - CRL appears to be appropriate. Distortion of the extensometer guide tube at the Phalen mine installation was likely to be attributed to excessive movements in the roof strata. Improvement in blocking the steel beams might reduce the risk of damaging the extensometer by rock movements.

- Install surface monitoring points on the structural members to detect deflections/distortion in the support system. This can be augmented by a levelling survey of the underside (Flange) of the roof and collar beams.

- Installation of other instrumentation such as load cells to measure the reactions at the end of collar beams would also be useful. It is understood that this type of instrumentation is being considered for monitoring rock load at an intersection in the Phalen Mine.

The above data would provide a basis for evaluating the appropriateness of the selected support system and for analysing the support requirements using

(a) an empirical approach which takes into account the rock mass characteristics

(b) an analytical approach which calculates the zone of loosening based on the rock mass strength, the in situ stress condition and the opening geometry

The results of the analysis would be useful for planning phase 3 of the program.

Phase 3

An experimental rock bolting program would be undertaken in this phase. This would include field trials of different rock bolt types and rock bolt patterns.

The objectives of the rock bolting program are to:

(a) identify the appropriate type of rock bolts for the coal measures strata and the performance of the bolts in response to possibly large rock movements

(b) study and demonstrate the performance of the bolts with and without supplementary supports such as straps or mesh panels or both

The trial rock bolting program could best be carried out in a full size experimental roadway or cross-cut where the roof condition is such that rock bolts are considered to be the appropriate alternative support system to steel sets. The rock bolting pattern used would be designed according to a stability analysis of the adit. Empirical design methods would also be referenced. The roof bolting system to be tested would comprise the following variations:

- rock bolts only
- rock bolts with straps (eg. w channels)
- rock bolts with straps and mesh

These supports should be installed close to the face to provide as much restraint to rock loosening as possible.

The displacements of the roof strata should be monitored by extensometers at the centre and the edges of the span. An array of extensometers will be required for each type of supports for systematic evaluation of the support performance.

Monitoring of the loads in rock bolts at intersections has been performed by the United States Bureau of Mines (Haramy and Hanna 1986). Similar instrumentation would be useful on the bolts for this phase of the program. Other devices such as a roof bolt tensioning indicators (manufactured by Good Year Canada Inc.) may provide a simple means of monitoring the rock bolt tension.

Phase 4

If the results from phase 3 are favourable the full scale implementation of a roof bolting program would be undertaken at a selected new roadway intersection. Prior to this phase, design of the rock bolting pattern and the supplementary support (if required) will have been completed.

The rock bolts would be installed according to the design and specified procedures. The performance of the support would be monitored by extensometers.

It is recognized that the rock conditions may be such that rock bolting may not be appropriate for use as the primary support. However, they can be strategically installed to increase the required steel set spacing or to reduce the effective span of an opening. Design and performance monitoring of this integrated support system would also be grouped under this phase of the program.

To extend the proposed experimental roof bolting study to the operating level, geological characterization of future mining areas based on a projection of the existing data is required. By means of an appropriate rock classification scheme, relative support requirements can be identified. As experiences are gained through gradual and systematic implementation of the roof bolting system, and performance monitoring, such a support system will be demonstrated to be a viable alternative or supplement to the steel set system.

6.0 CONCLUSIONS

A review of the support design and practice for roadway intersections in the underground coal mines in the United Kingdom, the United States, South Africa and Australia has been undertaken to examine the rock conditions, the types of support used and the support design procedure followed. Available statutory requirements relating to roadway supports are also summarized. In addition, the current roadway intersection supports used in the three producing mines in the Sydney coalfield were reviewed and an evaluation of a typical intersection was carried out to assess the rock loading on the support system by:

- back analysis from the estimated deflection of the structural beams at the intersection

- stability analysis of the roadway using numerical modelling to determine the potential overstressed zone at the roof of the roadway and intersection. Rock mass quality was taken into account in the analysis

- empirical method derived on the basis of rock mass quality

The conclusions derived from the review and evaluation are summarized below.

a. The tradition of using steel supports still dominates coal mining in the U.K., Europe and Canada. The roof support system in the United States on the otherhand is exclusively rock bolting. In South Africa and Australia, the current trend is to use rock bolts in the place of passive supports such as timber posts and steel arches. Steel sets are still used for poor

ground conditions. However, countries such as the U.K. and Canada are starting to implement experimental programs to study the viability of roof bolting.

b. The geotechnical design of either active or passive support systems for roadway intersections is still very much based on empirical methods. Analyses such as numerical modelling are used only on a few occasions but are generally limited to research oriented studies. Peng et al (1978) developed guidelines for roof support based on numerical modelling and calibrated using the depths of roof falls. Unal (1983) proposed a series of roof bolting patterns with two different types (mechanically anchored and resin grouted) of bolts for a range of roof conditions according to a rock mass classification scheme. However, more field data are required to correlate the support performance to the recommended support requirements.

c. The main roadway in the three operating coal mines in the Sydney Coalfield are generally supported by steel sets consisting of 80x100 mmx21 kg/m (for Prince and Phalen mines) and 114x127 mmx29 kg/m (Lingan mine) arch section spaced at 1 m centres. Timber lagging is used to block the steel set and steel corrugated sheets of 1.2x2 m are installed between the steel sets over the arched portion.

At the intersections, square work is typically used and the size of typical intersections is generally about 6x7m. A series of roof beams are generally used to transfer the rock load to collar beams which span across the slope. The sizes of the roof and collar beams vary between different mines (see Table 3.3). Noticeable deflections of between 50 to 70 mm in the collar beams have been observed in Lingan

mine, but only minor collar beam deflection was noted in the other mines.

d. The current structural design practice used by CBDC for the roadway and roadway intersection supports has been based on an assumed rock load on the support. The rock load is generally assumed to come from a triangular distribution of loosened rock with the height of the triangle equal to half of the span of the opening under consideration. The derivation of the design rock load is apparently independent of the geological conditions encountered at the intersections. This design philosophy tends to use the worst possible condition and disregards the need to restrict progressive loosening of the rock.

e. The potential failure zone at the roof of a typical roadway intersection in the Lingan Mine has been analysed by:

- back analysis from the estimated deflections of the collar beams and roof beams which are the main structural members for the roadway intersection support systems
- stress analysis of the openings pertaining to the intersection qeometry
- empirical design method

The salient points noted from the analysis are as follows:

- The depth of loosened rock obtained by back-analysing the collar beam deflections at a typical four-way intersection is about 0.6 to 0.8 m.

For such a depth of loosened zones, the corresponding Rock Mass Rating (based on the CSIR rock mass classification system) would range from 80-90. This is consistent with the observed condition of the rock and the corresponding estimated RMR values for the rock at the roof of typical roadway intersections.

- All the methods which take into account the rock mass characteristics give fairly consistent results. In comparison, the CBDC empirical design method tends to be conservative for the average condition at the Lingan Mine.

- The calculated depth of loosened zone indicates that the use of an active support system such as rock bolts is applicable to the rock and in-situ stress conditions for the cases analysed.

- A rational design method for roadway intersections should include a consideration of the rock mass characteristics, the in situ stresses, the geometry of the opening and a suitable stability analysis taking into account all these factors.

f. The rock conditions of coal measures strata in other coal mining areas are not significantly different from those encountered in the Sydney Coalfield in terms of its broad lithological characteristics. Various support types including active and passive support systems have been successfully used elsewhere in a range of coal mining environments. Some of these experiences may not be directly transferrable to the Sydney Coalfield because of site specific conditions but the experiences gained and the approaches used by others are useful references for the consideration of viable alternative support systems by the Cape Breton Development Corporation. The average roof condition at Lingan Mine is such

that roof bolting is a viable alternative support or a supplement to heavy steel sections for the main roadway intersections.

g. The increasing tendency to use heavier support in the operating mines in the Sydney Coalfield in response to the observed performance of the support (e.g. beam deflections) should be re-examined by considering: the need to justify the support used; the other factors which might have influenced the support performance but may not be directly related to rock loading; and the use of an alternative support system.

h. Recognizing the geological environment and the operating conditions in the Sydney Coalfield, a phased approach would be most appropriate for enhancing the current support design methodology and practices.

Phase 1
characterization of selected existing intersections

Phase 2
performance monitoring of a new intersection supported by current support systems

Phase 3
implementation and monitoring of an experimental program to demonstrate the performance of viable support alternatives

Phase 4
implementation and monitoring of a full scale intersection using the enhanced support system derived from phases 2 and 3

To extend the proposed experimental roof bolting study to the operating level, geological characterization of future mining areas based on projection of the existing data is required. By means of an appropriate rock classification scheme, relative support requirements can be identified. As experiences are gained through gradual and systematic implementation of the roof bolting system, and performance monitoring, such a support system will be demonstrated to be a viable alternative or as a supplement to the current steel set support.

7.0 ACKNOWLEDGEMENTS

Golder Associates would like to thank the Civil Engineering and Geological Department of the Cape Breton Development Corporation for their assistance in providing the background data for this project. Thanks are also due to Dr. K.Y. Haramy of the U.S. Bureau of Mines for his helpful discussion during the project. The assistance of Dr. T.R.C. Aston of the Cape Breton Coal Research Laboratory of CANMET throughout the study is also gratefully acknowledged.

Yours truly,

C.M.K. Yuen, P. Eng.

F. Miller

for J.W. Summers

CMK/FM/JWS/ma

REFERENCES: (cited in text)

Altounyan, P.F.R. and Bigby, D.N., 1988, "Roofbolting, A New Approach to Supporting Roadways" Colliery Guardian March 1988.

Gercek, H., 1982. Stability of Intersections in Room and Pillar Coal Mining, Ph.D. Thesis, The Pennsylvania State University, pp. 186.

Gerdeen, J.C. et al., 1977. Design Criteria for Roof Bolting Plan Using Fully Resin Grouted Nontensioned Bolts to Reinforce Bedded Mine Roof, Volume V - Synthesis and Design Criteria, U.S. Bureau of Mines Report, OFR 46(5)-80, pp. 100-104.

Hanna, K., Haramy, K., and Conover, D., 1985. Field Investigation of Roof and Pillar Stability in Coal Mine Intersections, Paper in Proc. Second Conference on Ground Control Problems in the Illinois Coal Basin, Southern Illinois Univ. Carbondale, IL., pp. 76-83.

Hanna, K., Conover, D., Haramy, K., and Kneisley, R., 1986a. Structural Stability of Coal Mine Entry Intersections - Case Studies, Paper in Proc. 27th U.S. Symp. on Rock Mech. Univ. of Alabama, pp. 512-519.

Hanna, K., Haramy K., Corover, D., and Dopp, D., 1986b. Effect of High Horizontal Stress on Coal Mine Entry Intersections Stability, Paper in Proc. 5th Conference on Ground Control Problems in Mining, West Virginia Univ., Morgantown, WV, pp. 167-182.

Hebblewhite, B.K. et al, 1986, Rock Mechanics and Stability of Excavations, Australian Coal Mining Practice, the Australasian Institute of Mining and Mettalurgy, Monograph series No. 12, pp. 148-161, Chapter 10.

Hoek, E. and Brown, E.T., 1980, Underground Excavation in Rock, Publisher: The Institution of Mining and Metallurgy.

Hocking, G., et al. 1976. Three-dimensional Elastic Stress Analysis of Underground Openings by the Boundary Integral Equation Method, Proc. Third Symp. on Engineering Application of Solid Mechanics, Toronto, pp. 203-217.

Kaneshige, O., et al. 1964. On the State of Stress Distribution Around the Crossing Gallery, (In Japanese), Journal of the Mining and Metallurgical Institute of Japan, Vol. 80 No. 914, pp. 659-669.

Mining Annual Review 1987 Pub: Mining Journal London June 1987.

Morris, C.J. and Newson, S.R., 1980, "Roofbolting Technology" Proc: Inst Mining Eng Mansfield 10 Feb. 1988.

Nisada, M. and Hondo, M., 1956. Photoelastic Investigation on the Stress Concentration due to Intersecting Holes, Proc. of the 5th Japan National Congress for Applied Mechanics, Science Council of Japan, pp. 131-184.

Obert, L. and Duvall, W.I., 1960. Design of Underground Opening in Competent Rock, U.S. Bureau of Mines Bulletin, No. 587, pp. 38.

Panek, L.A. and McCormic, J.A., 1973. SME Mining Engineering Handbook, SME of the American Institute of Mining, Metallurgical and Petroleum Engineers Inc., New York, Ivan A. Givens, Editor, Vol. 1, Section 13, pp. 13-/125-145.

Peng, S.S., Okubo, S. 1978, Roof Bolting Patterns at the Four-Way Entry Intersections, Paper presented at the 1978 AIME Annual Meeting, Denver, Colorodo, preprint no. 78-AM-5 pp. 18.

Peng, S.S., Okubo, S., 1978, Roof Bolting Patterns at Three-Way Entry Intersections, West Virginia University, Morgantown, Report No. 158.

Peng, S.S. 1980. Roof Falls in Underground Coal Mines, Technical Report, No. TR80-4, Dept. Min. Eng., West Virginia University, Morgantown, West Virginia, 44 pp.

Reuther, E.U. and Hermulheim, W., 1986, Rock Bolting in German Coal Mines-State of the Art and recent developments, 13th CMMI congress Mining

Richmond, A., et al 1986, Rock Mechanics and Stability of Excavations, Australian Coal Mining Practice, the Australasian Institute of Mining and Metallurgy, Monograph series No. 12, pp. 263-293, Chapter 18.

Stahl, R.W. 1962. Survey of Practices in Controlling Roof at Intersections and Junction in Underground Coal Mines, U.S.B.M., Information Circular 8113.

Steelman, J., 1986, Rock Mechanics and Stability of Excavations, Australian Coal Mining Practice, the Australasian Institute of Mining and Metallurgy, Monograph series No. 12, pp. 302-327, Chapter 20.

Unal, E., 1983. Development of Design Guidelines and Roof-Control Standards for Coal Mine Roofs, Ph.D. Thesis, Penyslvania State University, pp. 355.

Unrug, K. et al., 1984. Ground Stress and Roof Failure in Coal Mine Strata, Stability in Underground Mining II, Society of Mining Engineers, A.B. Szwilski and C.O. Brawner, editors.

Wagner, H. and Madden, B.J., 1984 "Fifteen Years Experience With the Design of Coal Pillars in Shallow South African Collieries: An Evaluation of the Performance of the Design Procedures and Recent Improvements" ISRM Symp: Cambridge UK September 1984.

Whittaker, B.N. and Scoble, M.J. 1980, "Rock Mechanics Instrumentation in Underground Ironstone Mining in the UK"

Wright, F.D., 1973. Roof Control through Beam Action and Arching, SME Mining Engineering Handbook, A.B. Cummins and I.A. Given, eds., SME of AIME, New York, Vol. 1, pp. 13-80 to 13-96.

Yuen, C.M.K., Boyd, J.M., and Aston, T.R.C., 1987, Rock-support interaction study of a TBM driven tunnel at the Donkin Mine, Nova Scotia, 6th International Congress on Rock Mechanics, editors, G. Herget et S. Vongpaisal, Montreal.

APPENDIX A

REPORT ON UNDERGROUND VISIT

TO LINGAN MINE

June 1988

871-1369

REPORT ON UNDERGROUND VISIT TO LINGAN MINE

1.0 INTRODUCTION

As part of the research study (DSS Contract No. 23440-7-9010/01-SQ) for the review and evaluation of main roadway intersections, an underground site visit to the Lingan Mine was undertaken by Messrs. John Summers and Clement Yuen. The visit was accompanied by Allan Harvey, Mining Engineer of CBDC for Lingan Mine, and Dr. T. Aston and Mr. G. Haslett of Canmet (CRL).

The visit was conducted by walking along the main roadways and cross cuts, starting at the number 6 cross cut. The route followed is marked in Figure A.1 attached. The main objective of the visit was to observe the general practice of the roadway support used in the mine with particular emphasis on the performance and support system for main roadway intersections. The observations made and the salient features noted at different areas visited are summarized in the following sections.

2.0 STRATIGRAPHY

A typical section of the roof and floor strata at Lingan Mine is shown in Figure A.2. In general, the coal seam (Harbour seam) is overlain by a thin mudstone seam (0.1 m thick approximately) followed by a sequence of siltstone, sandstone and limestone above. Below the coal seam the strata consist mainly of interbedded siltstone and mudstone with occasional, thin layers of sandstone. The mechanical properties of the rock are summarized in the same figure.

In Lingan Mine, the typical roadway dimensions are 5.1 m (17 ft.) wide x 3.6 m (14 ft.) high and they are developed with the bottom of the coal seam at the floor level. The roof of the roadway is therefore in either mudstone or siltstone depending on the local variations in the thickness of the mudstone and siltstone layers. Based on discussions with Mr. S. Forgeron, Chief Geologist of CBDC, the mudstone could vary from 0.2 to as much as 1.5 m.

3. OBSERVATIONS

The observations made at each area during the site visit are summarized in the following sections.

3.1 No. 6 Cross-cut from No. 4 to No. 1 Slope (Area 1)*

This development is flat topped and was originally cut to the full seam height. Support of the stone (mudstone) roof consists of RSJs set on short timber legs which have, in places, been set into pockets cut in the coal rib-side. Secondary cribs of timber with some stone packing had been added in an attempt to stabilize the supports following the failure of the pockets cut in the coal rib-side. Where falls have occurred, these were only to a height of approximately 0.5 m, and were mainly controlled by the stratification. Surface moisture was evident, as was iron staining associated with some bedding planes. Much of the timber was severely rotted and extensive rusting of the steel was evident.

3.2 No. 1 Slope between No. 6 Cross-cut and Turn (Area 2)

This arched development was measured as 17 ft. (5.2 m) wide and estimated as about 15 ft. (4.5 m) high. It was postulated that the design width was 16 ft. (4.8 m) and was deliberately overbroken to accommodate the steel arches. The steel arches comprised I sections, and joined in the crown, on the centre line, by bolted fish plates. There was evidence of timber lagging set above the crown of the steel sets, but the timber did not span the 3 ft. between individual arches. There was no noticeable deflection in the steel arches, but the timber lagging was severely rotted.

* Refer to Figure A.1

At some of the intersections along the slope, the roof has been ripped in the cross-cut to form the intersection, and falls have occurred at the brows thus produced. Most of the flat-topped RSJ supports are 3 1/2 x 3 1/2 in. (89 mm x 89 mm).

3.3 No. 1 Slope-Turn (Area 3)

Instrumentation has been installed at this position, requiring the drilling of one hole in each coal rib-side, and one hole in the crown. It was reported that there was no difficulty in drilling the crown roof hole, but that the rib-side holes tended to close up during drilling. The rib-side holes were drilled in coal, whilst the crown hole was drilled in stone (mudstone and siltstone). At the position of the turn, a brow formed by the development of a cross-cut had broken back to the bedding.

3.4 No. 8 Cross-cut to No. 2 Slope (Area 4)

Above this point the ground condition encountered is reported to be worse than that in other areas of the mine. This appears to coincide with the presence of water in the strata.

Below this point, an area of flat topped slope development was observed. Steel sets had been constructed at 12 to 15 in. spacing and no deformation of the steel work was observed.

3.5 No. 3 East Top (Area 5)

In this seam height development, half round timber poles had been fixed to the roof using mechanically anchored roof bolts with rectangular steel "dimple plates". A thin skin

of coal of 100 to 150 mm thickness had been left against the roof prior to the installation of the support. Subsequently the coal skin had frittered away to expose the mudstone of the immediate seam roof. This process had resulted in isolated bands or discs of coal forming, between the natural stone roof and the original mined roof, where supports were installed.

3.6 Bunker (Area 6)

This very high excavation was reportedly mined by "hard rock miners" and was supported by roof bolts, mesh and shotcrete. The observed mesh was weld mesh of 4 in. aperture to which a gunite/shotcrete layer had been applied. Some areas of the sidewall had been lined with monolithic concrete, but this was likely to be in order to secure the structural steel of the bunker, and associated motors and conveyors, rather than to support the rock walls. The roof was "lined" with corruaged sheets fixed to rails, which were in turn supported by long RSJ columns or by columns attached to the plant.

3.7 No. 8 East Top 3 Slope Ventilation Cross-over (Area 7)

This excavation has a total height of about 10 m and is square in plan. The brows formed by the excavated roof in the cross-cut showed a degree of blast shattering but, in some places, the half barrels of the shot holes were visible. Bolts were installed into the face of the brow, and wooden face boards were attached below the rock bolt plates. Timber lagging was also fixed to the surface by chains tensioned between bolts in the roof, and the arches of the No. 3 slope. Some arches, towards the edge of the cross-over, had been undercut during the blasting and were left with no foundation beneath the feet.

3.8 No. 7 West Bottom (Area 8)

This roadway is at the deepest point of the workings and is the maingate road for current longwall extraction. Immediately beyond the main Slope barrier pillar, the road is subject to "heavy roof". Excessive convergence was experienced. Steel arches showed excessive buckling, and the RSJs at No. 1 Slope intersection showed deflections larger than any others observed.

A change had then been made to yielding 'U' section arches manufactured by TH in Germany. These supports showed less deformation presumably as a result of the yielding capability of the arches.

3.9 General Observations at Roadway Intersections

Main roadway intersections were traditionally supported with rectangular frames comprising beams of about 8" x 8" which were, in turn, supported by columns of similar dimensions. Rock loosening was observed above the beams especially at the brow of the cross cut. The span of the cross cut at the intersection was estimated to be about 20 ft. (6 m).

Excessive deflections were observed in the cross beam at some cross cuts where significant rock loosening had occurred at the brow and timber laggings were absent.

At some intersections, however, the arched shape of the main roadway was partially retained. A straight RSJ was connected to the arch at the crown joint (by a fish plate) and supported by the cross beam at the cross cut to provide clearance for the intersection. Noticeable deflections were observed at the crown joint as this was the weak link of the support system.

3.10 Summary of Observations

(1) Falls of roof, where they could be observed, were generally of up to 1 m height and controlled by the bedding.

(2) Jointing is visible in some areas, and joints are seen to be sheared and slickensided in some of the argillaceous material. Joints do not appear to influence falls.

(3) In the deeper areas of the mine, the in situ stresses would be higher. In the rib-side, vertical tangential stress concentration would exist. Preliminary analysis of a typical longwall panel indicated that the stress concentration factor could be about 3.5 times the initial (pre-mining) vertical stress. This stress could well exceed the strength of the coal and mudstone layer at the rib-side. The weakened support in the rib-side will significantly affect the width of the loosened zone at the crown of the opening.

After the coal seam has been extracted and the longwall has advanced, the strata behind the longwall face will subside and large convergence is anticipated. Such strata subsidence will induce a tensile zone above the longwall panel. This tensile zone will enhance the opening of bedding surface and induce further loosening in the roof strata. At the edge of the longwall panel the gate road would be subjected to the influence of the collapsing longwall face resulting in "heavy" roof conditions compared with that in the main roadway. The yielding supports provided have accommodated larger deformation in the strata without excessive distortion of the sets.

(4) Where "massive" sandstone forms the crown of tunnels the failure mode appears to be fracturing, possibly accentuated by any blasting, rather than as a result of discontinuity control.

(5) In development of seam height a coal skin is often left against the mudstone which forms the immediate roof. This is a technique often used in coal mining where a mudstone is known to slake or weather rapidly upon exposure. Although there was no evidence seen of such slaking or weathering, the reasons for the protective coal skin being left may bear further investigation.

(6) The formation of the intersections tends to either produce brows of poorly supported material, or the arched profile results in unsupported haunches. Rectangular structural frames comprising heavy I-beams are currently being used for supporting main roadway and cross-cut intersections. Excessive deflections were observed in some of the beams across cross cuts due primarily to rock load brought about by loosening

at the brown and load transmitted from beams supporting the main roadway. A redesign of support must address these areas as it is here that the roof instability begins.

(7) Roof bolts in use are reported to consist of 2 lead mechanical anchors, with integral forged head, and are 5 ft. in length and 5/8 in. diameter. No failed bolts were seen although some lightly bolted falls involved the anchor having been pulled out of the hole. This would tend to infer that falls do not usually reach 1.5 m (5 ft.) in height.

4.0 CONCLUSIONS

The observations made during the site visit have indicated that in the main roadway, the depth of loosening in the strata at the crown of the roadway opening is generally less than 1.5 m under average rock condition (where mudstone overlies the coal seam and is also located at the roof) where sandstone was encountered at he roof, very limited rock loosening/fracturing was noted. The critical aspects of roadway supports are summarized as follows:

(1) There is an advantage in using the natural stratification as the final roof especially at intersections.

(2) Active rock reinforcement should warrant consideration as it will reduce progressive loosening of the strata by developing the interlocking effects between the strata before loosening takes place. In the main roadway and roadway intersections, the use of fully column grouted rock bolts in combination with straps and mesh (if required) should be beneficial. A trial section for the use of rock bolts as the primary support system should be considered. This section should be inspected at regular intervals and monitored with instruments to assess the overall performance and efficacy of the support.

In the gate roads where heavy ground condition occurs as a result of the influence of the longwall extraction method, a combination of rock bolts and steel set support could be used. With the rock bolts providing the primary support, the steel set section would likely be lighter.

The reuse of the gate road for subsequent adjacent longwall extraction requires that the gate road be maintained during the second stage mining operation. The use of monolithic packing systems has been suggested by Singh and Atkins (1984) for reinforcing the side support for the gate road.

APPENDIX B

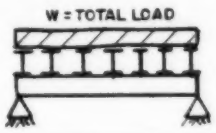
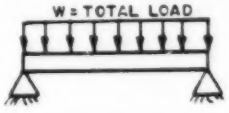
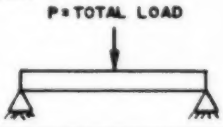
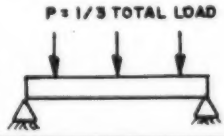
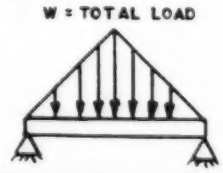
FORMULAE FOR MAXIMUM DISPLACEMENT AND BENDING STRESS CALCULATIONS

June 1988

871-1369

Table B.1

FORMULAE FOR MAXIMUM DISPLACEMENT AND BENDING STRESS CALCULATIONS

Load Distribution (simply supported beam)	Maximum Bending Moment (M) (N•m)	Maximum Displacement (δ) (m)	Maximum Bending Stress (σ) (MPa)
Closely Spaced Point Load 	cWL	$k \cdot \frac{WL^3}{EI}$	$\frac{M}{S}$
Uniformly Distributed Load 	$\frac{WL}{8}$	$\frac{5}{384} \cdot \frac{WL^3}{EI}$	$\frac{M}{S}$
Single Point Load at Mid Span 	$\frac{PL}{4}$	$\frac{1}{48} \cdot \frac{PL^3}{EI}$	$\frac{M}{S}$
3 Point Load at quarter Span 	$\frac{PL}{2}$	$\frac{19}{384} \cdot \frac{PL^3}{EI}$	$\frac{M}{S}$
Triangular Load 	$\frac{WL}{6}$	$\frac{1}{60} \cdot \frac{WL^3}{EI}$	$\frac{M}{S}$

L = total beam span (m)
E = Young's Modulus (Pa)
I = moment of inertia (m^4)
S = elastic section modulus (m^3)

W = Total Load (N)
P = Point Load (N)
M = Maximum Bending Moment (N•m)

Notes: 1.

n	c	k
7	.1224	.0128
8	.125	.0128
10	.125	.0129

where n = number of point load
c = bending moment factor
k = displacement factor

if n>10, assume uniformly distributed load

2. maximum bending moment, displacement and bending stress are calculated at mid span of simply support beam

June 1988

871-1369

APPENDIX C

ANALYSIS OF ARCH STABILITY IN MAIN ROADWAYS AT LINGAN MINE

June 1988

871-1369

ANALYSIS OF ARCH STABILITY IN MAIN ROADWAYS AT LINGAN MINE

C.1 Introduction

The main roadway in the Sydney Coalfield serve as major transportation and ventilation routes. The steel arch is the principal system of roadway support used by the Cape Breton Development Corporation. The choice of the support and the support spacing was based on the capacity of the arch to accept the rock load and to keep the openings usable.

According to the previous visit to the mine (section 3.1.2) and the pictures on plates 1 to 9 furnished by CBDC, the arches in the main roadways across the mine appear to be stable, with no noticeable signs of overstressing and excessive deflection.

This section presents the analysis of the slope support located close to the principal intersections according to the range of rock condition at the four and three way intersections.

Lingan Roadway Support

At Lingan mine, the steel arches for the main slope roadways comprise rigid arches of 5.5 X 3.7 m. Usually, eight steel spacers of 1 m length are used to separate the arches.

Table C.1 summarizes the typical roadway support for Lingan, Prince and Phalen Mines. At Lingan mine, the

arches are composed of two steel sections jointed in the centre by generally a wrap-around fishplate. The foot plates of the arches are usually set on the intact floor rocks. Wood laggings are installed at the arch inflection point to stabilize the support. The space between the roof and the crown of the arch is generally fill with wood lagging. On each side of the arch some steel sheet or wood lagging are installed to prevent the rockfalls from the sidewall. The general installation of typical arches is presented on plates 1 to 9.

To evaluate the arch stability, the results of the boundary element analysis described in Section 4.0 were used to determine the approximate extent of loosened rock over a typical arch in the main roadway. The loosened zone was approximate by a uniform load distributed on the arch and an analytical solution (Choquet 82-88) was used to calculate the corresponding centre deflection and the maximum stress of the arch. The results were compared with the field observations.

Boundary Element Analysis

According to the results of stability evaluations of a typical intersection by back calculation and numerical modeling, the depth of loosened rock at the intersection ranges from 0.6 to 1.0 m for the average rock conditions encountered in the main roadway. The depth of loosening along the roadway is anticipated to be shallow because of the narrower span.

Based on the same range of rock classifications used for the intersections (RWR ranging from 65 to 90) the potential overstressed zone and the equivalent uniform load on the arch was analysed using the Boundary element program. The results are summarized in Table C-2 and

they indicated that the loosened zone ranges from 0.3 to 1 m distributed over 40° to 50° angle from the vertical centre line of the arch.

To analyse the stresses and deflections in a typical roadway arch section, the analytical solution proposed by Choquet (1982, 1986) was used. The following assumptions were used in the solution:

- the steel arch is composed of two semi-circular sections jointed by a fishplate at the crown
- the joint is considered to be a strong as the steel section used for the arch
- the points of reaction (see sketch table C-2) are located on each side at 50° from the vertical centre line of the arch section
- the rock load is applied as a uniformly distributed load along the circumference of the arch. The angle subtended at the centre of the arch could vary (see sketch table C-2)
- A stiffness was assumed for the lagging at the reaction points and for the supporting strata at the contact of the foot plate

Table C-3 summarizes the properties of the arch section and the input parameters for the analyses. The deflections and tensile stresses calculated at the crown of the arch for different loads and loading angles are shown in Table C-4 and plotted in Figure C-1.

Based on these results, the arch would not be overstressed when the uniformly distributed load is less than 1.0 m thick of loosened rock. The maximum

tolerable crown deflection is about 80 mm. However, when the loading is increased, the extent of the loading (θ_1) would be severely restricted. For example, the yield stress of steel (300 MPa) would be reached when a loosened zone of about 1.2 m is distributed more than 15° on each side of the apex of the arch.

Based on visual observation during the site visit, there is no noticeable overstressing of the arches nor is there any excessive flattening of the arches. The inferred maximum deflection of the arches is approximately 20 to 40 mm. Hence it is estimated that the following loading configurations are possible:

a) 0.6 m of loosen rock distributed over 15°

b) less than 1.2 m of loosen rock distributed over 10°

These results are generally consistent with the average rock mass rating estimated for the rock and the rock mass strength parameters assumed.

REFERENCES

Choquet, P., 1982. A failure criterion of steel arch support for the interpretation of in situ monitoring results, CIM special Volume NO. 30, The Canadian Institute of Mining and Metallurgy, 14th Canadian Rock Mechanics Symposium, Peter Baumgartner editors.

Tincelin, E., Choquet, P., 1982. Un critere de rupture du soutenement par cintres pour l'interpretation des mesures enregistrees in situ: validation par essai d'ecrasement en laboratoire, Tunnels et ouvrages souterrains, revue bimestrielle, N.51.

Choquet, P., 1986. Dimensionnement des Cintres de Soutenement du Tunnel en Tenant Compte de la Presence des Assemblages des Elements et de la raideur des garnissage, paper presented in the International Congress on Large Underground Openings, Frenze, Italy, Volume II.

Choquet, P., 1988. Professor at Universite Laval, Quebec, personnal communication.

TABLE C[illegible]

SUMMARY OF THE SYSTEM OF ROADWAY SUPPORT

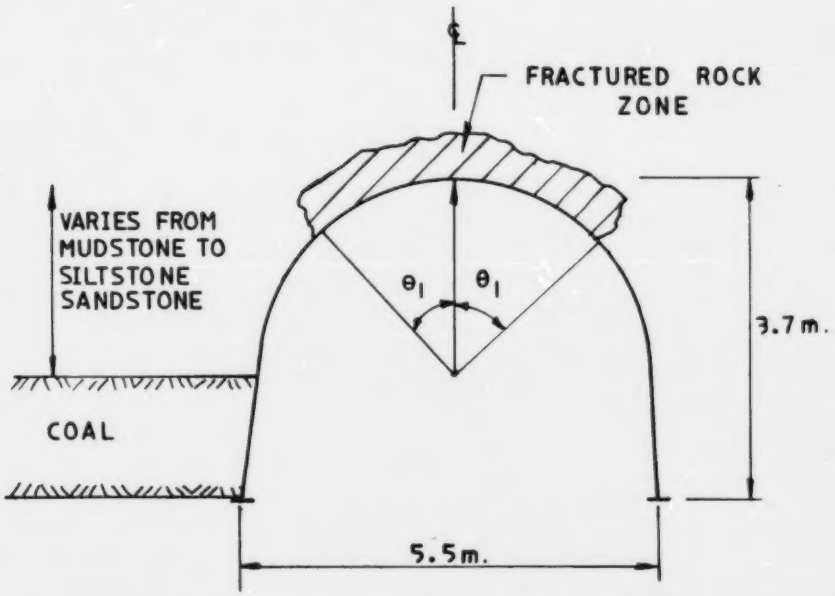
(Based on CBDC data)

LOCATION	TYPE	WEIGHT	CROSS SECTION AREA	SIZE	SPACING	NO. OF SPACERS	TYPE OF FISHPLATE	LAGGING	ROADSIDE PACKING
LINGAN									
7 WEST TOP	RIGID	29 kg/m	38 cm^2	114 x 127 mm	4.3 x 3.4 m	1 m	6	WRAP AROUND	WOOD	HARDWOOD CHOCKS
7 WEST BOTTOM	YIELDING	29 kg/m	36.96 cm^2	125 x 148 mm	4.3 x 3.4 m	1 m	7	CLAMPS	WOOD	HARDWOOD CHOCKS
10 EAST TOP	RIGID	29 kg/m	38 cm^2	114 x 127 mm	4.3 x 3.4 m	1 m	6	WRAP AROUND	WOOD	HARDWOOD CHOCKS
10 EAST BOTTOM	RIGID	29 kg/m	38 cm^2	114 x 127 mm	4.9 x 3.7 m	1 m	7	WRAP AROUND	WOOD	MONOLITHIC PACKING PLUS HARDWOOD CHOCKS
11 EAST TOP	RIGID	29 kg/m	38 cm^2	114 x 127 mm	4.3 x 3.4 m	1 m	6	WRAP AROUND	WOOD	HARDWOOD CHOCKS
11 EAST BOTTOM	RIGID	29 kg/m	38 cm^2	114 x 127 mm	4.9 x 3.7 m	1 m	7	WRAP AROUND	WOOD	MONOLITHIC PACKING PLUS HARDWOOD CHOCKS
SLOPES	RIGID	29 kg/m	38 cm^2	114 x 127 mm	5.5 x 3.7 m	1 m	8	WRAP AROUND	CORR. SHEETS	
PRINCE									
6 WEST TOP	STEEL BOOMS	25 lb/ft	6" x 6"		4.3 m	1 m	7	CLAMPS		
6 WEST BOTTOM	STEEL BOOMS	25 lb/ft	6" x 6"		4.3 m	1 m	7	CLAMPS		RETREAT SYSTEM - NO ROADSIDE PACKS
SLOPES	RIGID	21 kg/m	26.4 cm^2	80 x 100 mm	4.9 x 3.7 m	1 m	7	WRAP AROUND		
PHALEN									
1 WEST TOP	RIGID	21 kg/m	26.4 cm^2	80 x 100 mm	5.5 x 3.7 m	1 m	6	WRAP AROUND		RETREAT SYSTEM - NO ROADSIDE PACKS
1 WEST BOTTOM	TRAPEZOID	29 kg/m	38 cm^2	114 x 127 mm	4.9 x 2.7 m	1 m	7	HOLLYBANK JOINT		
1 EAST TOP	TRAPEZOID	29 kg/m	38 cm^2	114 x 127 mm	4.9 x 2.7 m	1 m	7	HOLLYBANK JOINT		
1 EAST BOTTOM	STEEL BOOMS	29 kg/m	38 cm^2	114 x 127 mm	4.6 x 2.7 m	1 m	7	HOLLYBANK JOINT		
SLOPES	RIGID	21 kg/m	26.4 cm^2	80 x 100 mm	5.5 x 3.7 m	1 m	8	WRAP AROUND		

Golder Associates

TABLE C-2

SUMMARY OF THE BOUNDARY ELEMENT
ANALYSIS OF A TYPICAL SLOPE SECTION



RMR	Description	Rock Mass Parameter m	s	Equivalent Thickness of the Fractured Rock (d) (m)	Loading Angle θ_1 (deg)	Total Equivalent Uniform Load kN/m
65	Mudstone	3.0	0.02	1.0	50°	27
80	laminated siltstone	5.2	0.1	0.3	45°	8.1
90	siltstone and sandstone	6.9	0.32	0.3	40°	8.0

June 1988

871-1369

TABLE C-3

INPUT PARAMETERS USED IN THE EVALUATION OF THE ELASTIC DEFLECTION AND TENSILE STRESS AT THE CENTRE OF THE ARCH

Lingan Arch

Type: Rigid (127 X 29)
Weight: 29 kg/m
Sections: 3.8×10^{-3} m^2
Size: 5.5 x 3.7 m
Spacing: 1 m

No. of Spacers: 8
Type of Fish Plate: Wrap around
Lagging: Corrugated sheets and wood

***Arch used in the Calculation**

Type: Rigid (M130 x 28.1)
Weight: 28.1 kg/m
Sections: 3.6×10^{-3} m^2
Size: 5.5 x 3.7 m
Type of Fish Plate: Considered as continuous
Lagging: Wood

Input Parameters Used in the Calculation

Young modulus of steel (E) = 206×10^6 kN/m^2

Moment of inertia (I) = 1.0×10^{-6} m^4

Elastic section modulus with respect to X-X (S) = 15.8×10^{-5} m^3

Specified minimum yield point of steel (F_y) = 300 MPa

Ultimate strength of steel (F_u) = 450 - 620 MPa

Wood stiffness (K_1) = 10,000 kN/m

Rock stiffness (K_2) = 4.9×10^6 kN/m

Arch radius (R) = 2.8 m

Rock Load (P) (along arch circumference)	17	34	51	68	(KN/m)
Equiv. thickness of loosened rock	0.6	1.2	1.8	2.4	(m)

NOTES:

1. *Equivalent beam registered in the Canadian Steel Handbook. This beam is not available in Canadian Mills, but was used for the purpose of calculation only.

2. Input parameters used in formulae given by Choquet (1986)

TABLE C-4

ELASTIC DEFLECTIONS AND TENSILE STRESSES
CALCULATED AT THE CENTER OF A
TYPICAL ARCH AT LINGAN MINE

Equivalent Thickness of Loosened rock (m)	Total Equivalent Uniform Load (kN/m)	Loading Angle θ_1 (deg.)	Tensile Stresses At Centre (MPa)	Deflection At Centre (m)
0.6	17	5	57.10	.0142
0.6	17	10	107.7	.0280
0.6	17	20	189.8	.0532
0.6	17	30	246.8	.0731
0.6	17	40	278.3	.0857
1.2	34	5	114.2	.0284
1.2	34	10	215.5	.0560
1.2	34	20	379.7	.1063
1.2	34	30	493.5	.1463
1.2	34	40	556.7	.1714
1.8	51	5	171.3	.0426
1.8	51	10	323.2	.0840
1.8	51	20	569.5	.1595
1.8	51	30	740.3	.2192
1.8	51	40	835.0	.2570
2.4	68	5	228.4	.0568
2.4	68	10	430.9	.1120
2.4	68	20	759.3	.2127
2.4	68	30	987.1	.2922
2.4	80	40	1113.4	.3427

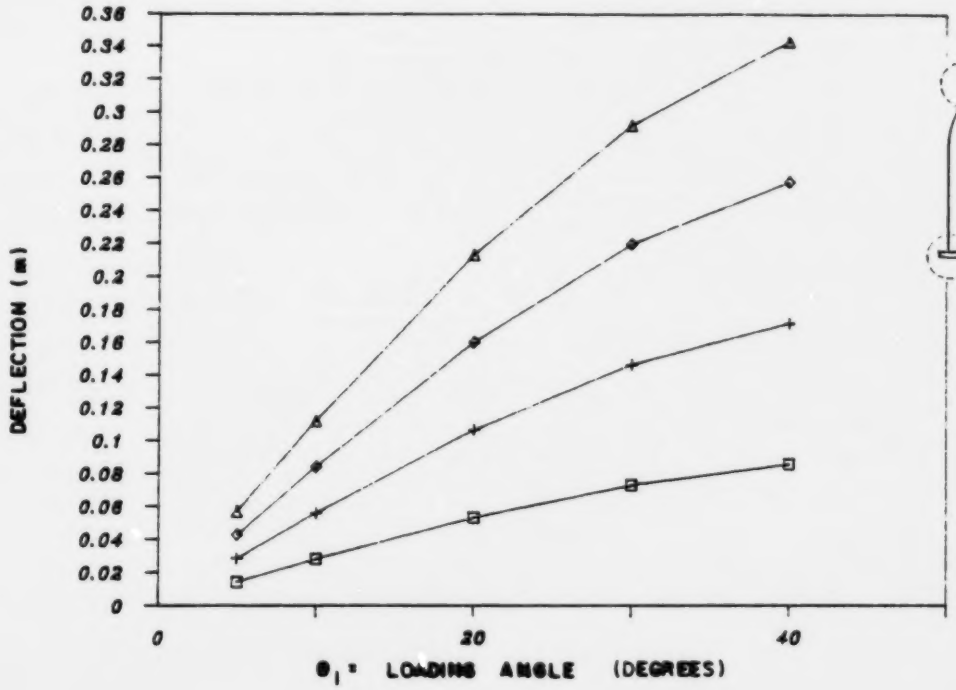
NOTES:

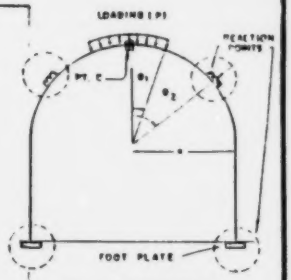
Calculations based on Choquet (1986)

PLOT OF THEORETICAL ELASTIC DEFORMATION AND TENSILE STRESS OF AN ARCH UNIFORMLY LOADED

FIGURE C.1

ELASTIC DEFLECTION AT THE CENTRE OF THE ARCH (PT. C)



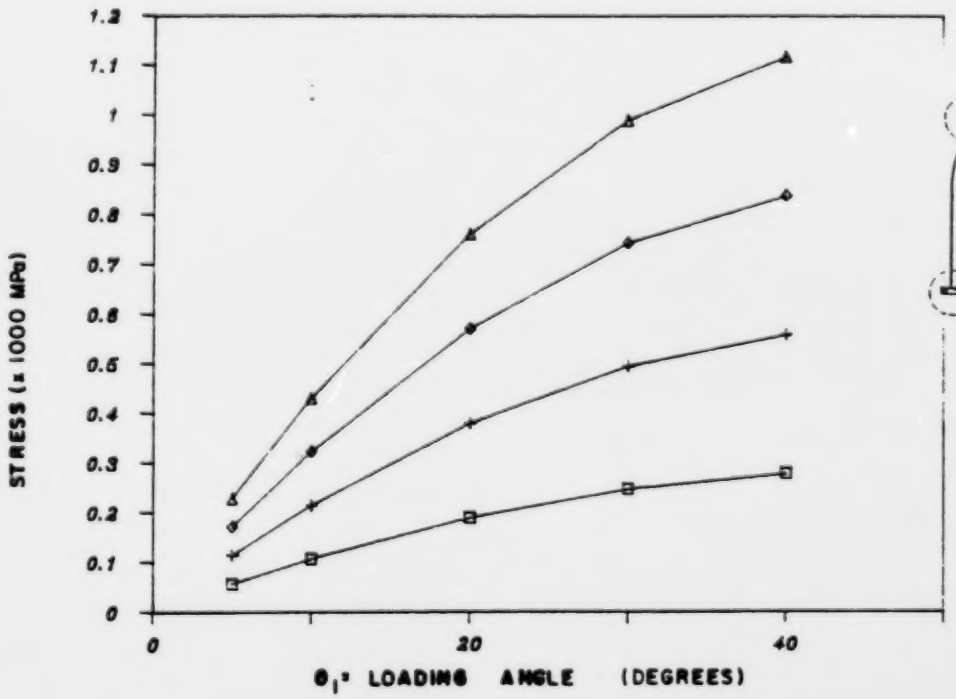


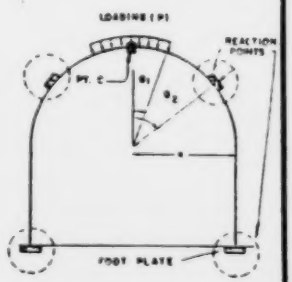
θ_1 = VARIABLE

θ_2 = 50°

REACTION POINT

ELASTIC TENSILE STRESS AT THE CENTRE OF THE ARCH (PT. C)





θ_1 = VARIABLE

θ_2 = 50°

REACTION POINT

□ P = 17 kN/m. (0.6m) + P = 34 kN/m. (1.2m)

◇ P = 51 kN/m. (1.8m) △ P = 68 kN/m. (2.4m)

Date APRIL 27/88

Project 871-1369

Golder Associates

Drawn D.M.

Chkd F.M

FORM PRODUCED JUNE 1988

Form G.A.-D-4m (metric)

MAPS
NOT
FILMED

CARTES
OMISES